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THE APPLICATION OF THE RESULTS OF CERTAIN SOIL TESTS TO THE DESIGN OF AIRPORT RUNWAYS

REINARD W BRANDLEY

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THESIS

THE APPLICATION OF THE RESULTS OF CERTAIN SOIL TESTS TO THE DESIGN OF AIRPORT RUNWAYS

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of the
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Master of Science

by

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Signature of Author

Thesis
1946
#1.

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Introduction

Previous to World War II the largest plane that was built was about as heavy as an ordinary car. Design procedures for runways for such planes paralleled those for the design of highways. With the approach of the war the planes started to grow in size rapidly until today some of the large bombers in use weigh as much as 75 tons, giving a 75,000 pound wheel load. Plans are now on paper for some transport planes with wheel loads of 110,000 pounds or more.

The Department of Transport (Canadian Government) in the construction of a runway have not had in their specifications that the thickness of base should be designed for but have followed the following general procedure:

The area was first cleared.

The area to be covered by the finished runway was stripped of all loam and deleterious material.

The subgrade surface was brought to grade by cut where necessary and fill with suitable clay where necessary.

Preparation of the subgrade usually consisted of scarifying to a depth of 6 to 10 inches followed by compaction. In some cases pit run gravel was ploughed into the top 4 to 8 inches of the subgrade and then compacted.

The base course generally consisted of 4 to 9 inches of compacted gravel base or plant mixed stabilized base.

A Bituminous hot mix pavement of 2 to 4 inches covered with a seal coat of sand and asphalt generally formed the runway surface. In some cases a blotter surface was laid.

With the great increase in wheel load for the planes the old design procedures rapidly became obsolete and it became necessary to follow other design methods.

The problem was tackled by many different men and departments with varying results. A comparison of the results obtained by applying 5 different published formulae is published in the Journals of Asphalt Technology (March-June 1943 and September-October 1943) by Dr. N.W. McLeod. In this comparison there is a large discrepancy between the results. Dr. McLeod is of the opinion that the Gray and Goldbeck formulae and the California method are too conservative while the Housel method leads to under design.

The United States Engineering Department (U.S.E.D.) who have constructed numerous airport runways throughout the war have used the California method of design.

The Department of Transport have been unwilling to accept the prevailing methods of design of airport runways. As it is essential that with the heavy wheel loads that some scientific method of design be used, a research program was undertaken on a number of the airports in Canada during the summer of 1945. The results from this program will be used in this paper.

In the early days of runway construction it was the opinion of the authorities that the runways had to take the heaviest loadings and that the taxi strips and warm up areas were not subjected to nearly so heavy loads. In later years it has been found that the opposite is the case and that the warm up areas are subjected to the heaviest loads. The warm up areas are damaged considerably by the oil and gasoline leaking from the engines of the planes. This oil and gas disolves the asphalt in the surface and thus gives an entrance to the base for moisture and a starting point for complete failure. Consequently for design many authorities have recommended increasing the wheel load designed for by some factor. Twenty-five percent has been recommended by the U.S.E.D. Concrete for these areas has also been recommended.

On an airport runway complete failure seldom occurs. It may, however, occur on the warm up strips and parking areas. When one speaks of failure of a runway he is referring to settlement of different portions of the runway, by excess loading or repetitive loading, making the runway unusable or so it cannot be comfortably used for landing and take off. Differential settlement of a runway will cause failure of the surface which leads to complete failure. The Asphalt people state that 0.5 inches of settlement produces failure of the surface. This figure will be used in the following portions of the paper to determine the bearing capacity of the base course, and base course plus surface.

The aim of this paper is to correlate the data from certain laboratory soil tests with that obtained from field loading tests in order that one can establish the strength of a subgrade from a laboratory test. The results obtained from this will then be applied to obtain design curves for base course plus surface thicknesses of runways for different wheel loadings.

TESTS RUN ON THE SOIL

I. Field Tests & Sampling

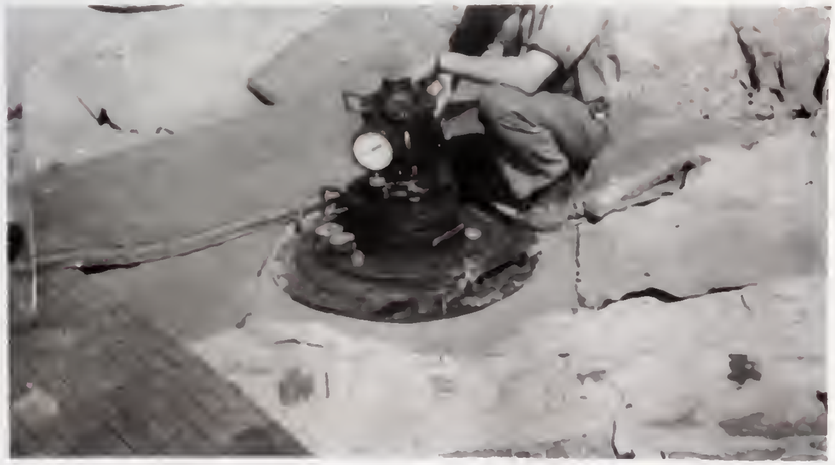
Some of the tests run in the field were for the purpose of establishing the strength: of the subgrade materials, of the base course, of the base plus surface. These tests included: The Surface Load Test (load bearing test on asphalt surface), The Base Course Load Test (load bearing test on the base surface), The Subgrade Load Test (load bearing test on subgrade surface), the Cone Penetration tests and Housel Pentrometer test. Density and moisture content of the subgrade base and pavement were taken to determine what state the materials were in at the time the other tests were run.

Samples were taken, both undisturbed and otherwise, so laboratory tests could be run on the same type of material as the field tests were run on.

A description of the field tests and sampling operations follows.

(a) Load Bearing Test

The location at which the test was to be run was spotted. The plates were then set in place as shown in the following photo.



The 30" diameter plate in this test was placed first. On top of this was placed the 24" diameter plate, then the 18" diameter plate, then a series of 12" diameter plates and finally the 50 ton hydraulic jack. These plates and jack were so placed that a vertical line through the centre of the jack went through the centre of each plate. Each plate was 1 inch thick except for the 30" plate which was 1¼"

For the surface test the plate was set in a thin layer of Plaster of Paris. For the base course and subgrade tests the plate was set in a thin layer of screened sand. (Sand passing the No. 20 sieve).

The truck loaded with about 30 tons was next driven over the plates in such a way that the verticle through the centre of the jack passed through the centre of gravity of the load.

An 18 foot beam was next passed under the truck with its axis perpendicular to the long axis of the truck. The beam rested on the pavement on either side of the truck through suitable supports. Its purpose was to hold the Aimes dials which were used to measure the settlement of the plate.

The dials were set on opposite ends of a diameter of the bottom plate about one inch from the edge. Enough load was applied by means of the hydraulic jack to give about 10/1000 inches deflection of the plate, to set the plate, and the dials were set at zero at this load. The load was applied until it was thought from watching the dials that by the time the plate had finished settling the total deflection would amount to approximately 0.05 inches.

When this load was reached the load on the jack was recorded and dial readings were taken at the following times after the complete load had been applied:

One, two, three, five, seven, ten and fifteen minutes and then every five minutes until the rate of deflection of the plate was 1/1000 inches per minute. The amount of deflection obtained when the rate of deflection was 1/1000 inches per minute was recorded as the deflection obtained after the first application of the first load.

The hydraulic jack was released thus letting the load off the plates and dial readings taken every minute until the rate of recovery was equal to 1/1000 inches per minute. The total deflection from start of test to this time was recorded as the settlement obtained after first release of first load.

The process of loading and releasing was repeated 6 times for the same load and the values of deflection and settlement recorded.

The load was then increased until it was thought the deflection after the first application of this load would be approximately 0.18 inches. The application and release of this load was repeated 6 times and the values for deflection and settlement recorded.

The load was next increased to give an initial deflection after the first application of approximately 0.35 to 0.40 inches. The application and release of this load was repeated 6 times also and the values for deflection and settlement recorded.

The first application of each load was continued for at least 15 minutes and succeeding applications of load were continued until the difference between each of two successive deflection readings was 1/1000 inches per minute or less.

The results were plotted as shown in the following sample results.

Airport - Grande Prairie
Runway - #11 NW-SE
Station - 34 + 80
Offset - 33' R
Datum - CL
Date - June 29, 1945
Type Test - Subgrade Load Test

10,000#

	1st	2nd	3rd	4th	5th	6th
Deflection	.0525	.0550	.0590	.0610	.0620	.0620
Settlement	.0145	.0170	.0200	.0225	.0245	.0260

20,000#

	1st	2nd	3rd	4th	5th	6th
Deflection	.1400	.1510	.1540	.1610	.1640	.1700
Settlement	.0590	.0680	.0710	.0760	.0800	.0830

40,000#

	1st	2nd	3rd	4th	5th	6th
Deflection	.3400	.3700	.3930	.4050	.4200	.4280
Settlement	.1680	.1970	.2180	.2270	.2350	.2420

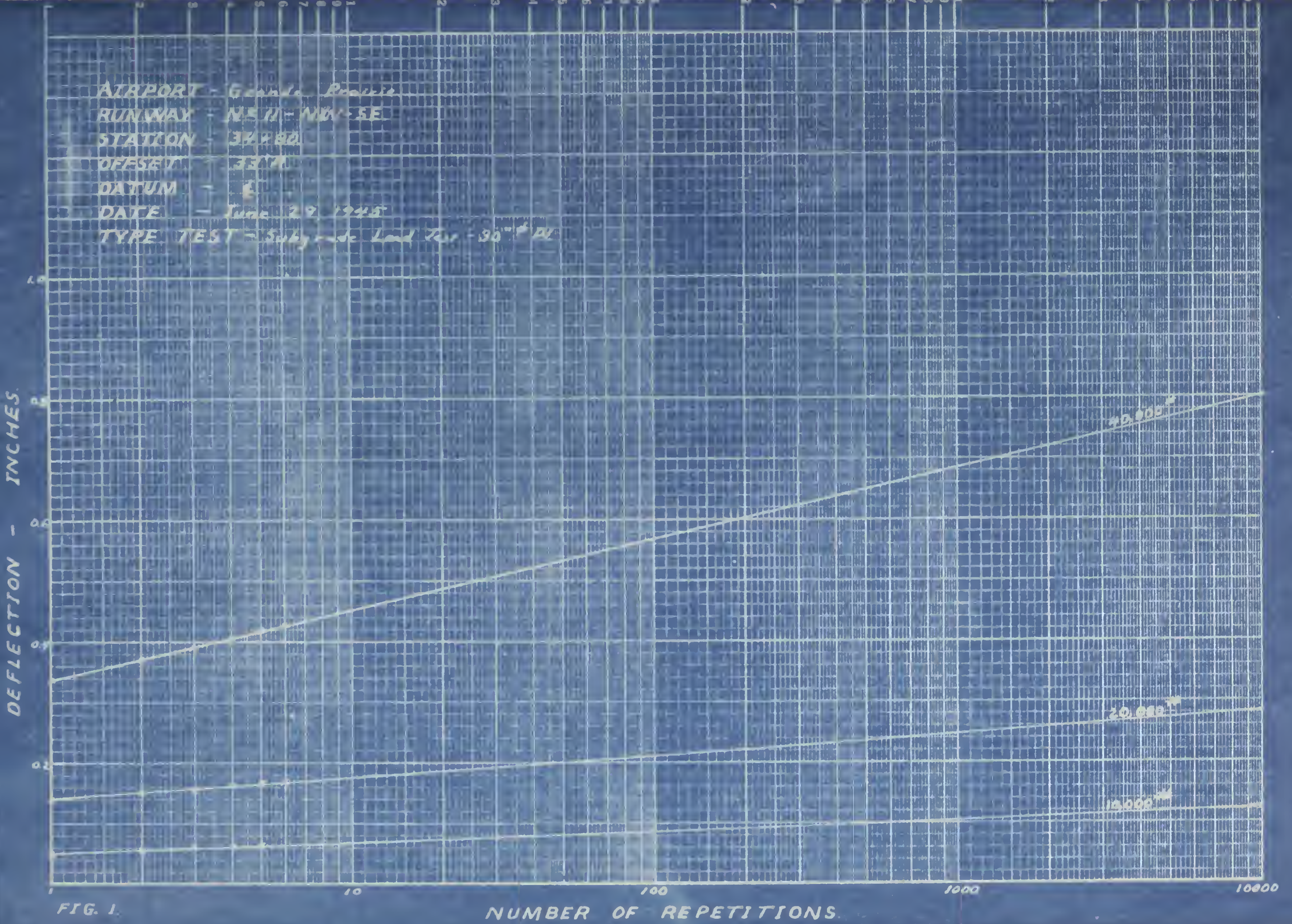
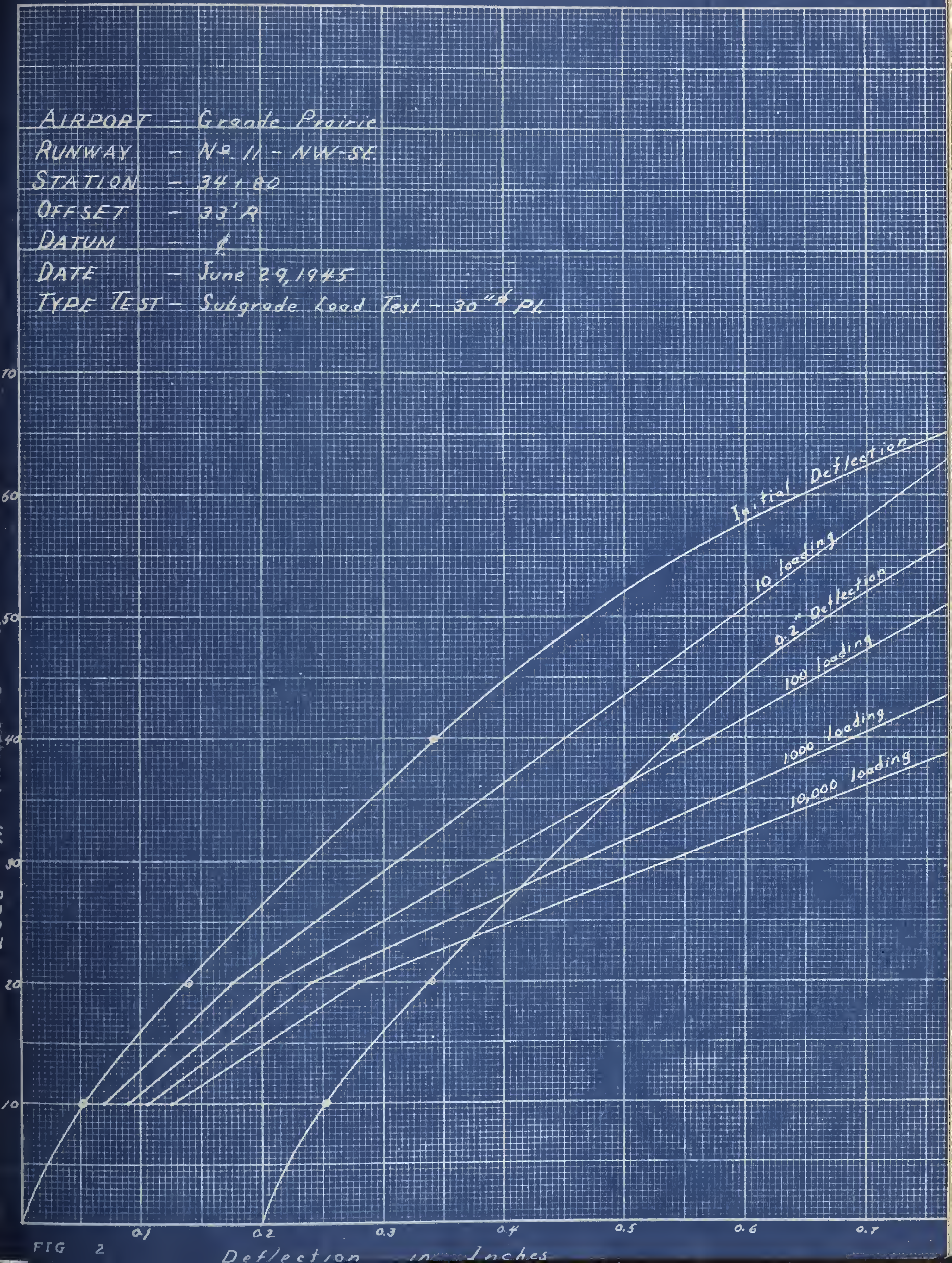

AIRPORT - Grande Prairie
RUNWAY - N.E.11 - NW-SE
STATION - 34+80
OFFSET - 33 Rt.
DATUM - ℄
DATE - June 29, 1945
TYPE TEST - Subgrade Load Test - 30" Dia.
40,000#
20,000#
10,000#
1.0
0.8
0.6
0.4
0.2
1
10
100
1000
10000
DEFLECTION - INCHES
NUMBER OF REPETITIONS

FIG. 1.



AIRPORT - Grande Prairie
RUNWAY - Nº 11 - NW-SE
STATION - 34+80
OFFSET - 33'R
DATUM - ℄
DATE - June 29, 1945
TYPE TEST - Subgrade Load Test - 30"⌀ PL

FIG 2

The six values of deflection for each load were plotted against Log (number of repetitions)(Fig.1) and straight lines were drawn through these points out to 10,000 repetitions. To justify the extending of these lines load tests were run to one hundred applications of load at each field. The straight line relationship was found to hold. However, as this is not used in this paper, no more will be said about it.

On the arithmetic plot of load vs deflection (Fig. 2) the deflection after the first application of each load was plotted giving the Initial Deflection Curve. The 0.2" Deflection Curve was located by plotting the curve which had 0.2 inches more deflection at each load than did the Initial Deflection curve.

The 10,100, 1000 and 10,000 loading curves were plotted by picking the results from the Deflection vs Log (number of repetitions) Curve at the corresponding number of repetitions. These curves were drawn in as straight lines as shown on the graph. In this paper the only curve that is used is the initial deflection curve.

(b) Cone Penetration Tests at the undisturbed test location were run at 0, 6 & 12 inches below the top of the subgrade. The test was repeated at each layer until 3 checks were obtained. Test as outlined in the A.S.T.M. "Proceedings for Testing Soils".

(c) Housel Penetrometer Test at the undisturbed test location was run for 0-6, 6-12, & 12-18 inches below top of subgrade and the shear samples obtained were taken to the field laboratory. Test conducted as recommended in "A Manual of Soil Science Applied to Subgrade and Base Course Design" by Dr. N.W. McLeod.

(d) Thickness and classification of all layers overlying the subgrade was measured and recorded.

(e) At the undisturbed location

Density sample (approximately 1000 grams) and a 75 pound sample of asphalt pavement was taken.

Moisture samples (approximately 1200 grams kept in glass sealers) density (using Ottawa Sand) and a 75 pound sample of base course was taken.

Moisture samples (approximately 1000 grams kept in glass sealers) density (using Ottawa Sand) and 75 pound samples of subgrade

at 0-6 inches below top of the subgrade
at 6-12 inches below top of the subgrade
at 12-18 inches below top of the subgrade

were taken.

Moisture samples (approximately 1000 grams kept in glass sealers) were obtained for 2 foot intervals up to 10 feet below the top of the subgrade, by augering.

(f) Two C.B.R. undisturbed samples 6" in diameter by 6" long were taken from each undisturbed sampling location at each of 0-6" and 9-15" below the top of the subgrade.

(g) Under all load test locations 60# samples and moisture content samples of: base course, and subgrade 0-6, 6-12 & 12-18" below the subgrade surface were taken.

II. Laboratory Tests

All laboratory tests unless noted otherwise were run in the Soil Mechanics Laboratory at the University of Alberta.

The laboratory tests were run for four main purposes:

(a) To classify the soils

(b) To determine the subgrade strength

(c) To obtain the moisture density relations

(d) To determine moisture contents of the base course and the subgrade.

(a) Classification of soils

In nature there are many different kinds of soils such as sandy soils, silty soils, organic clays, inorganic clays, etc. It is a known fact that each type of soil will have different characteristics such as permeability, bearing capacity etc.

The tests used for classification purposes are: The Limit Tests, Specific Gravity, and Mechanical Analysis.

(b) Determination of Subgrade Strength

The method of finding the strength of a subgrade material is a hotly contested point. If the answer were known there would be no need for this investigation. There are however, several tests that have been developed for the purpose of obtaining the strength of a subgrade material. The California State Highway officials devised a punching shear test called the California Bearing ratio test for obtaining the strength of a subgrade material.

The Triaxial Compression Test gives a value as to shearing strength and modulus of deformation of a subgrade material. The Triaxial Test run on the clays was a quick test and thus values of angle of internal friction obtained are by no means the true values but the values for modulus of deformation and cohesion are reliable.

The Consolidation test is ordinarily run to give amount and rate of settlement of a soil mass subjected to a load. However, in this investigation this is not of too much importance. It was hoped to utilize the values of swelling pressure and permeability and certain soil constants, as the slope of the virgin branch of the pressure void ratio curve, that can be derived from this test.

In the field laboratory Housel shear tests were run on the samples obtained from the Housel Penetration Test. This is a direct shear test where the soil is acted on in double shear. The test was conducted as outlined in "A Manual of Soil Science Applied to Subgrade and Base Course Design" by Dr. N.W. McLeod.

(c) There is another test which cannot be included as a classification or strength determination test. This is the moisture density test or Proctor Compaction Test. The purpose of this test is to set standards of maximum compaction that can be obtained by a material and at what moisture content the soil can be compacted to this maximum. It was run here to compare field densities with maximum density as obtained by standard test. The test run was the Modified Proctor; The reason being that with modern compaction methods one can obtain higher densities than can be obtained by the Standard Proctor Test.

(d) Moisture content - The natural moisture content of all subgrade and base materials as they existed in the field were taken in order that other test data could be better correlated.

A description of the tests used follows:

1. Limits

The tests as outlined in the A.S.T.M. book "Procedures for Testing Soils" for: Liquid Limit, Plastic Limit, Plasticity Index, Flow Index, Shrinkage Limit, Shrinkage Ratio, and Specific Gravity from the Shrinkage test were conducted on each 75 pound sample taken from the undisturbed location.

2. Specific Gravity

Tests for finding the true specific gravity of the grain particles, both coarse and fine, were run by using water as outlined in the Harvard University's "Notes on Soil Testing for Engineering Purposes".

3. Mechanical Analysis

Sieve and hydrometer tests on each sample from the undisturbed location for grain size distribution as outlined in the Harvard University's "Notes on Soil Testing for Engineering Purposes".

4. California Bearing Ratio

Standard test as laid down by the California State Highways Officials with the modification that the test was run on one end of the sample, three quarters of an inch was cut from that end, the sample soaked, and then the test rerun on the same end as the first test, was run on one of the undisturbed samples from each depth of each sampling location.

5. Consolidation Test

A consolidation test was run on a sample taken from the other of the 2 undisturbed samples taken from each depth of the undisturbed sampling location. The test was conducted as laid down in the Harvard University's "Notes on Soil Testing for Engineering Purposes".

6. Triaxial Test

Quick Triaxial tests were run on samples taken from the same undisturbed sample as the consolidation test samples.

The test was conducted as laid down in the Harvard University's "Notes on Soil Testing for Engineering Purposes".

7. Moisture Density

Modified Procter Compaction - same test procedure as outlined in the A.S.T.M. book "Procedures for Soil Testing" with the modification that the soil is compacted in 5 layers, each layer being compacted by 25 blows of a 10 pound hammer falling 18 inches.

8. Moisture Contents

Determination of moisture content of the base course and each subgrade moisture sample that was taken in the field were run in the field laboratory.

Moisture content determination was run on the base by using 1000 gram samples and on the subgrade materials by using 200 gram samples. The samples were oven dried for 18 to 24 hours at 110° ± 5° C.

9. Density of the Asphalt was also determined in the field laboratory by the water displacement method.

Data and Materials

The vast amount of data used by the author for this investigation was obtained from the Department of Transport from a research program which was carried out on the various airports in the summer of 1945. The laboratory test results were obtained from the Department of Civil Engineering which, under the direction of Prof. R.M. Hardy, conducted the tests for the Department of Transport.

The author worked in the field during the summer of 1945 for the Department of Transport on this research project.

Classification of Soils

A method of classification of soils is the Plasticity Chart, which is a plot of Plasticity Index vs Liquid Limit. If a series of such tests on soils from the same geological origin are plotted on this plasticity chart the points will all fall on a straight line. The location on the straight line for such soils is dependent on the grain size, the coarser materials being located at the lower end of the line and the finer materials at the upper end.*

The plasticity chart therefore can readily be used for classification purposes. It has been divided roughly into sections as shown in Figure 3. Each section has the name of the soil type inserted.

Therefore to classify the soils used in this investigation a plasticity chart will be drawn for the soils from each airport and called "Soil Classification Curves". (Figures 4 - 7).

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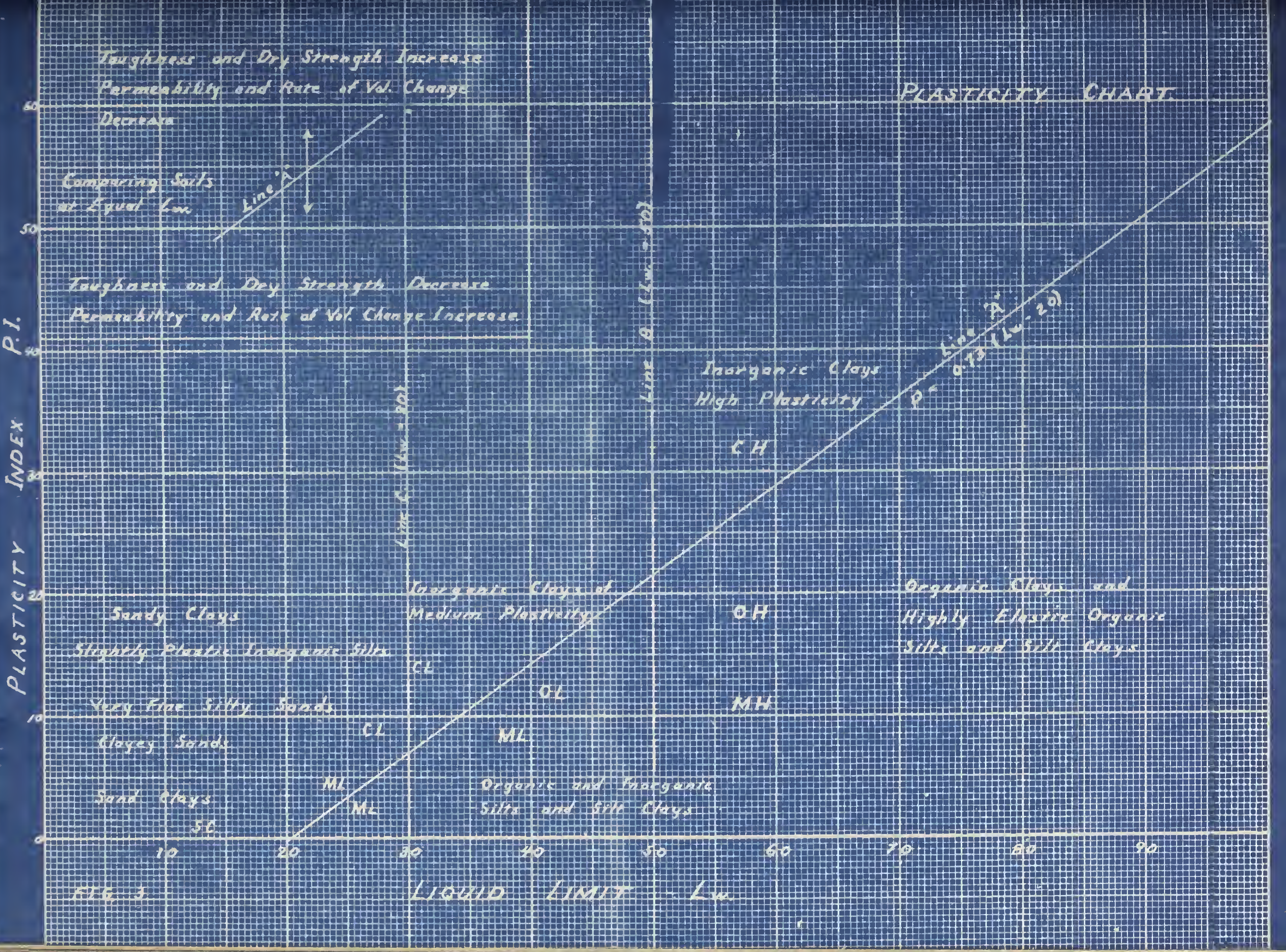
* D.K. Terzaghi - Engineering Institute of Canada Journal - December 1944.



PLASTICITY CHART

FIG. 3

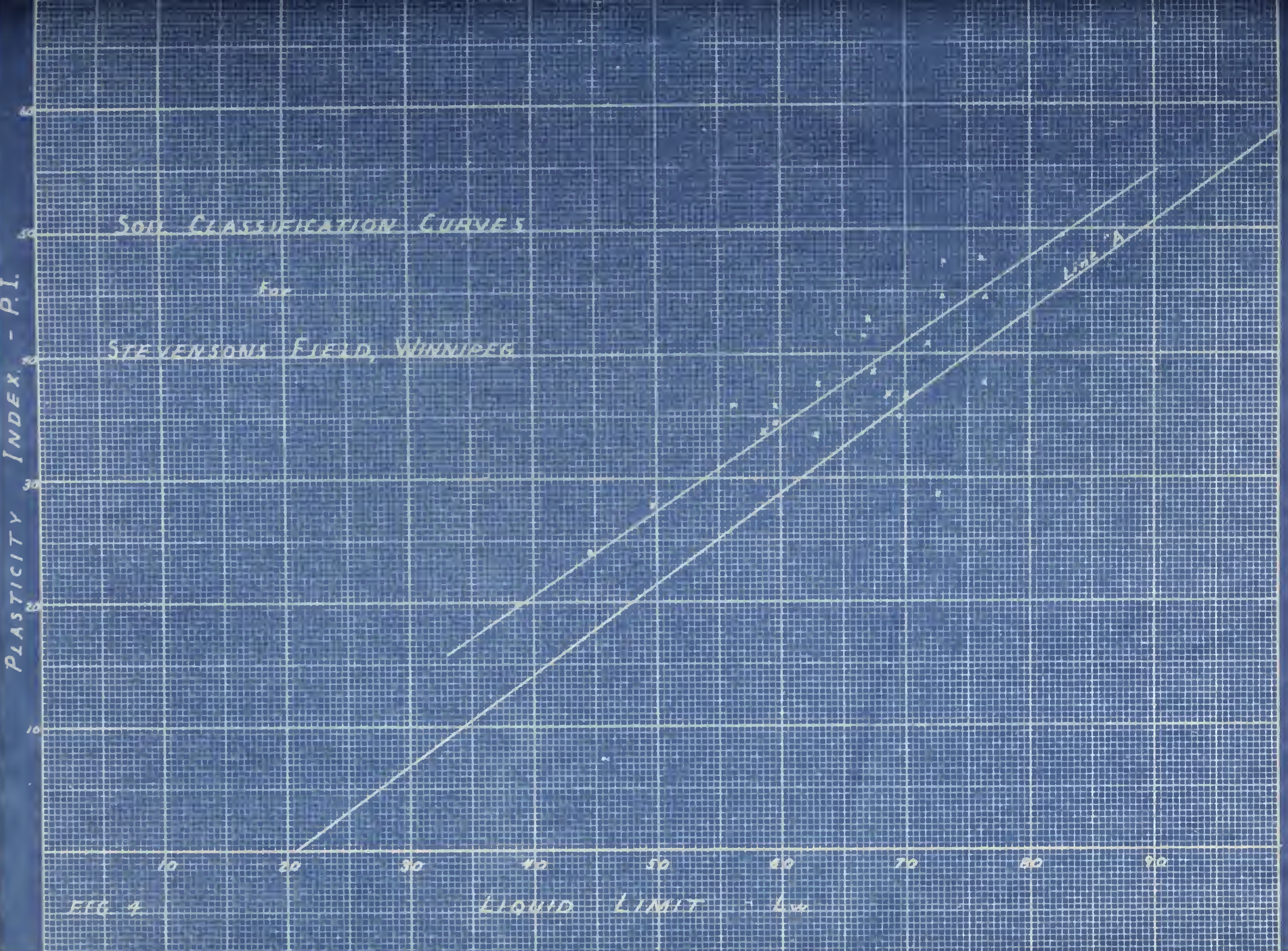
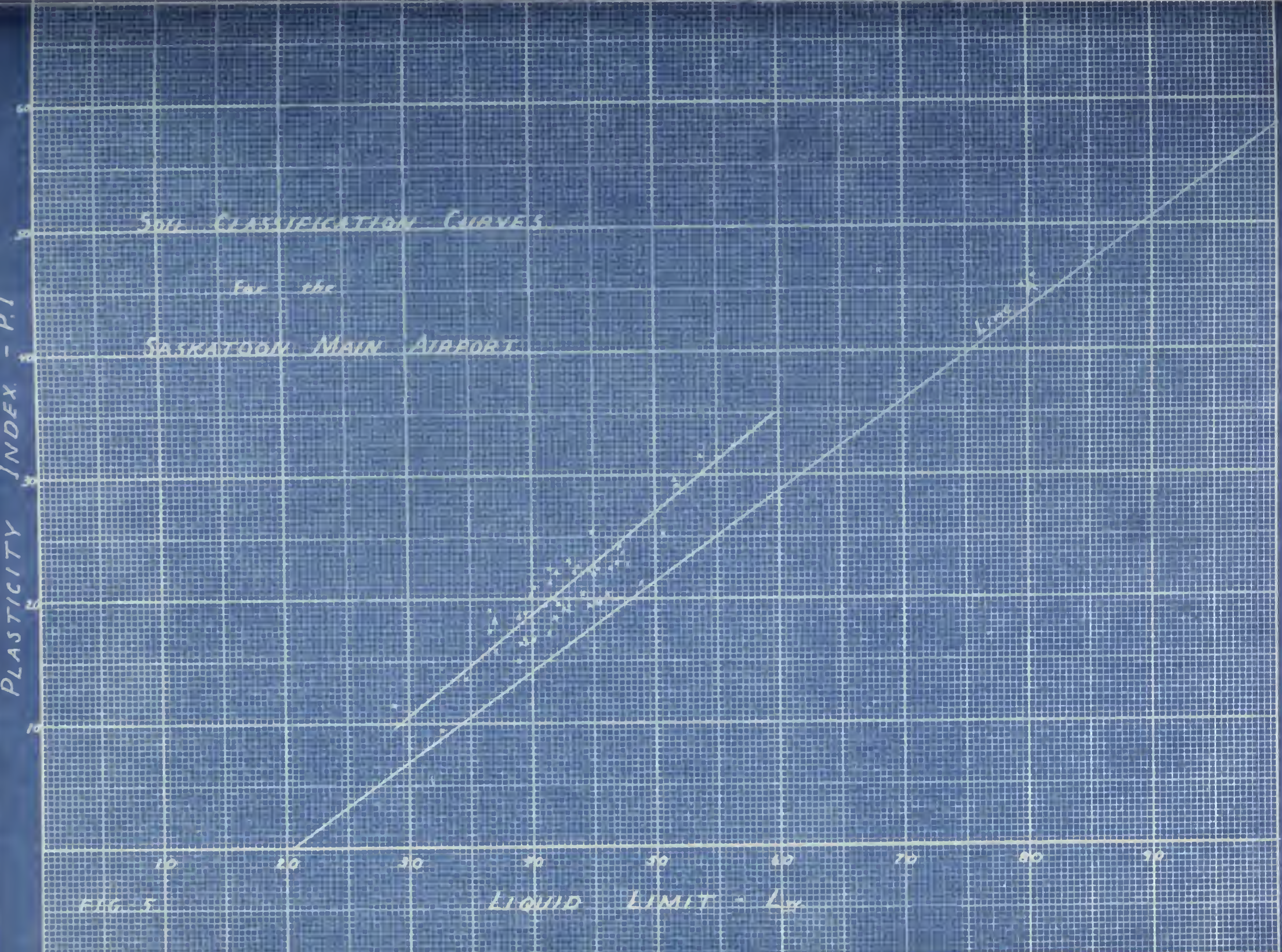


FIG. 4

SOIL CLASSIFICATION CURVES

for

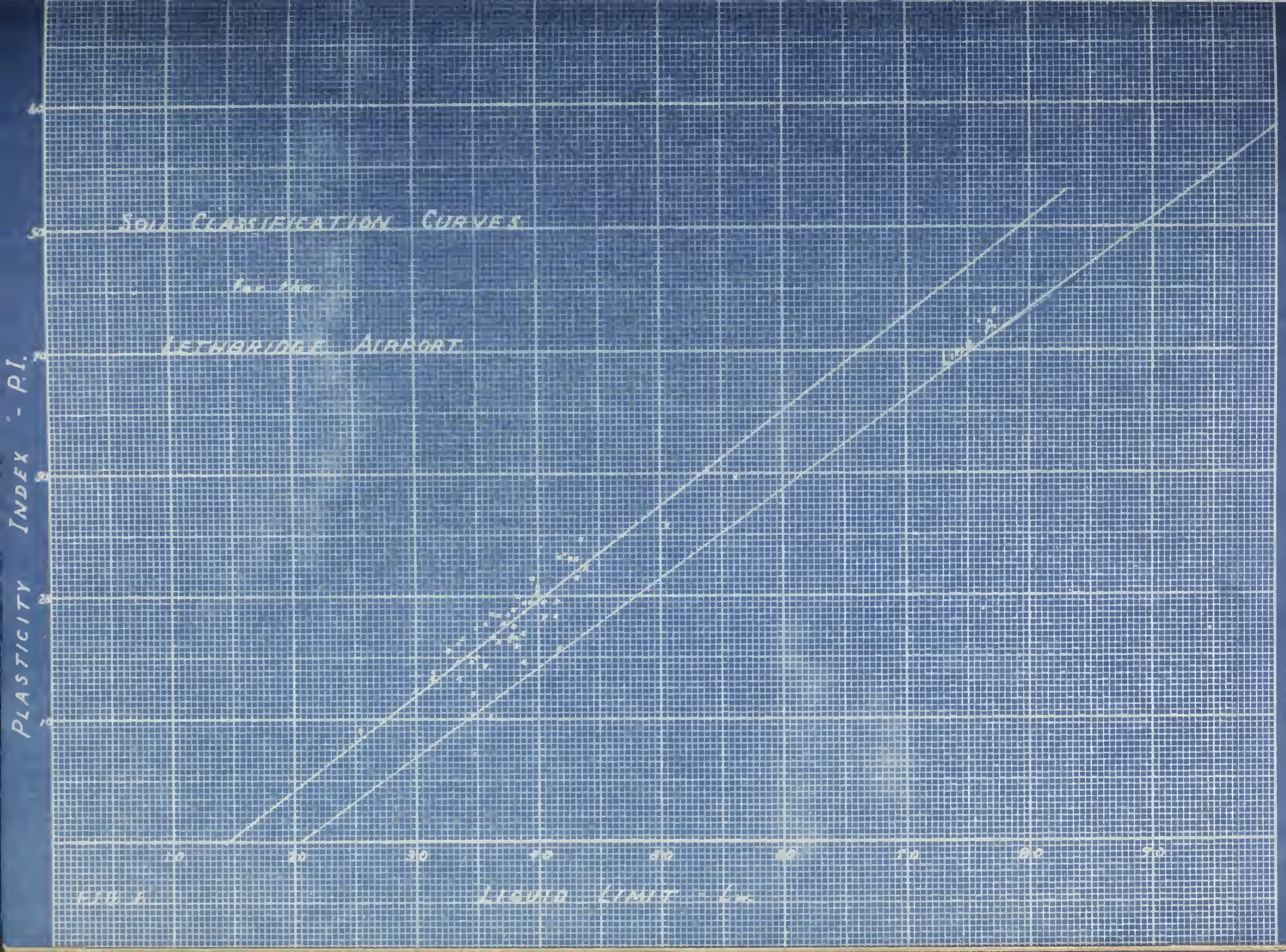
STEVENSONS FIELD, WINNIPEG


SOIL CLASSIFICATION CURVES

For the

SASKATOON MAIN AIRPORT

FIG. 5

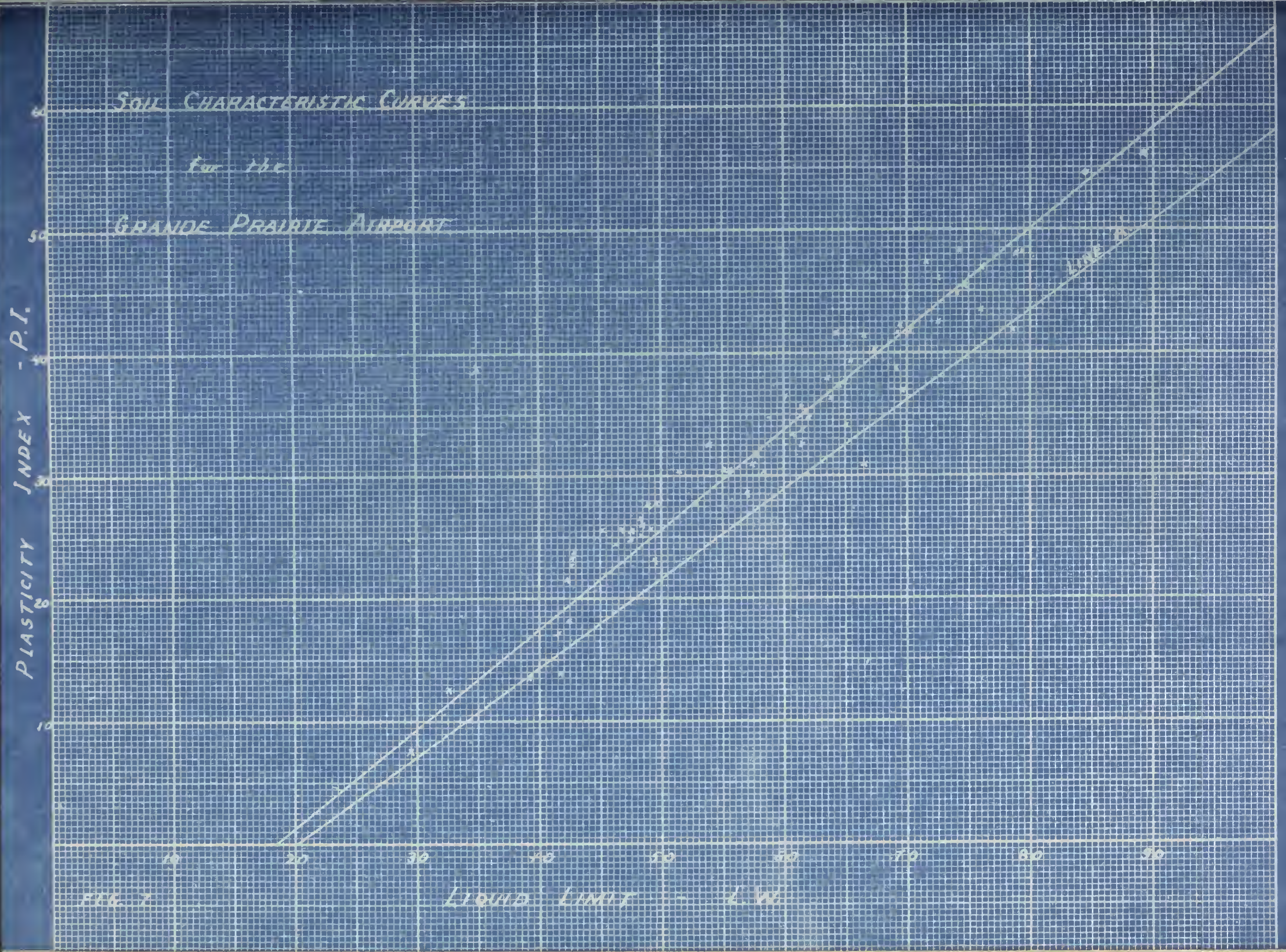


SOIL CLASSIFICATION CURVES

for the

LETHBRIDGE AIRPORT

FIG. 6

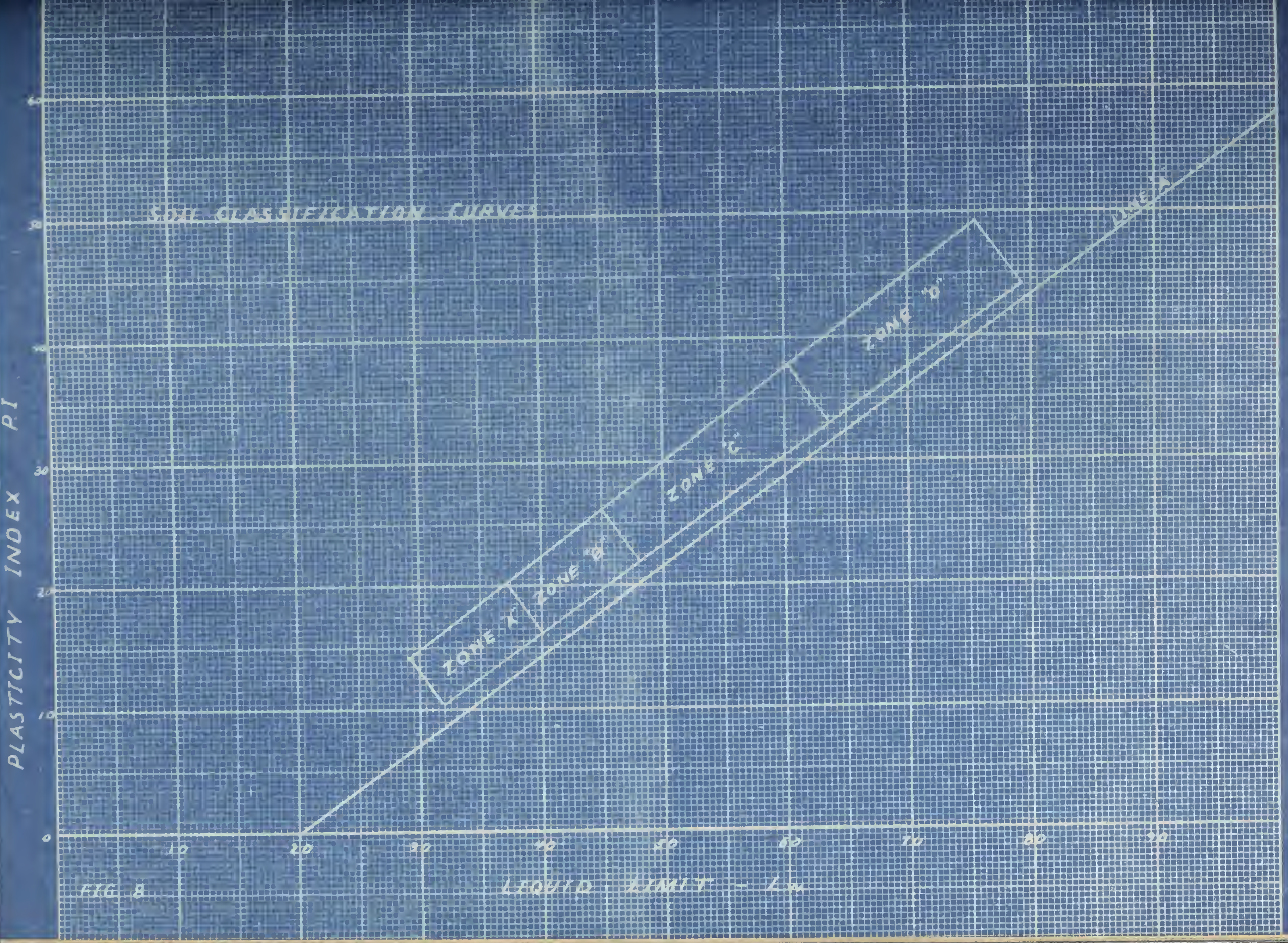


SOIL CHARACTERISTIC CURVES

for the

GRANDE PRAIRIE AIRPORT

FIG. 7



SOIL CLASSIFICATION CURVES

FIG. 8

A study of the classification charts shows that the plot of plasticity index vs liquid limit gives a rather good straight line relationship. This was expected as on the plains the soils in a small area are very apt to be of the same geological age. Therefore it can be assumed that for all the work on each airport one is dealing with the same type of soil.

The classification chart can now be divided into different zones as shown in Figure 8. More work will have to be done on other types of soils to enable the complete zoning of the chart.

The classification chart has been divided into 4 zones here:

Zone A - Lethbridge subgrade soils
Zone B - Saskatoon subgrade soils
Zone C - Grande Prairie subgrade soils
Zone D - Winnipeg subgrade soils

From the foregoing work it appears that the first step in the design of an airport runway is to obtain a reasonable number of samples of the subgrade to be used and run Liquid Limit and Plastic Limit Tests on them. In this way by referring to the Classification Chart of Figure 8 the zone into which the soil in question falls could be determined.

In the remaining part of this paper the work will all be done for each zone. The author is not of the opinion that a definite relation between the subgrade strength as available in the field and as found from the laboratory tests can be obtained for all types of soil in general, but possibly such a relation can be obtained for each individual type of soil.

It was thought that one could obtain some relationship for classification purposes from the values of liquid limit and shrinkage limit. A plot was therefore made (Figure 9) of Liquid Limit vs Shrinkage Limit. The wide distribution of results on this plot indicate that there is little significant relation between these two limits. Therefore no further attempts will be made to use these values as the classification charts of Plasticity Index vs Liquid Limit prove to be quite satisfactory.

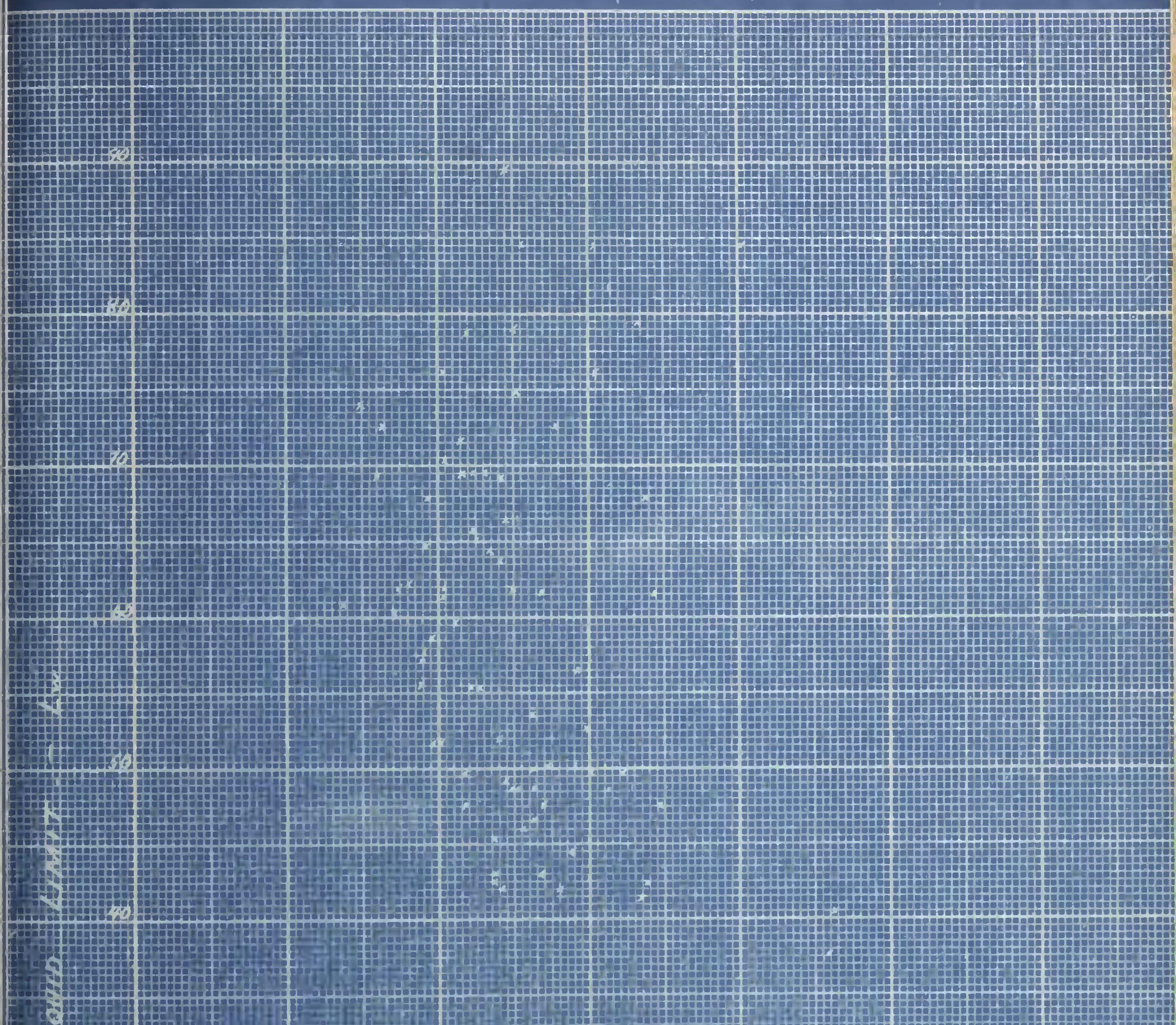

LIQUID LIMIT - L.L.
90
80
70
60
50
40
30
20
10
5
10
15
20
25
SHRINKAGE LIMIT - S.L.
LIQUID LIMIT v SHRINKAGE LIM
for the
GRANDE PRAIRIE AIRPORT

FIG. 9.

Bearing Capacity of Subgrade Materials

In this work it will be assumed that the field load tests on the surface of the subgrade will give the bearing capacity of the subgrade. The problem then is to correlate the laboratory data with the field data such that from the laboratory tests one can determine the bearing capacity of the subgrade material.

It is therefore essential to first establish the type of subgrade failure that was obtained when the field load test was run.

1. Shear Failure

The load required on a shallow circular footing of radius R to produce a shear failure is given as follows[*]

$$Q_D = \pi R^2 \left(1.3\, c N_c + \gamma D_f N_q + 0.6\, \gamma R N_\gamma \right).$$

where $$N_c = \cot\phi \left[\frac{a_\theta^2}{2 \cos^2 (45 + \frac{\phi}{2})} - 1 \right].$$

$$N_q = \frac{a_\theta^2}{2 \cos^2 (45 + \frac{\phi}{2})} .$$

$$N_\gamma = \tfrac{1}{2} \tan\phi \left(\frac{K_p}{\cos^2\phi} - 1 \right).$$

$$a_\theta = e^{\left(\frac{3}{4}\pi - \frac{\phi}{2}\right)\tan\phi}$$

$=$

e = base of naperian logarithms

ϕ = angle of internal friction

K_p = coefficient of passive earth pressure for a surface footing; cohesion = 0.

δ = angle of wall friction equals ϕ

D_f = depth of burial of base of footing

γ = unit weight of soil

N_c, N_q, N_γ can be obtained from published charts[**]

---

[*] Dr. K. Terzaghi - Theoretical Soil Mechanics - Chapter VIII SS 46 & 47

[**] Dr. K. Terzaghi - Theoretical Soil Mechanics - Figure 38, Page 125

If the soil is loose or very compressible, the bearing capacity factors N must be replaced by the values N^1.

In order to compute the corresponding values N_c^1, N_q^1 & N^1 one must replace the values of ϕ & C in the above equations by C^1 & ϕ^1.

$$C^1 = 2/3\ C$$

$$\tan \phi^1 = 2/3 \tan \phi$$

These N^1 bearing capacity factors can also be obtained from published charts.*

In the type of subgrade load test which was run in the field, $D_F = 0$. As the subgrade material was clay the test was run too fast to have much internal friction developed so it will be assumed that $\phi = 0$.

Therefore $Q_D = \pi R^2 (1.3\ C\ N_c + 0.6\ \gamma\ R\ N_\gamma)$

$N_c = 3/2\pi + 1 = 5.7$

$N_\gamma = 0$

Therefore $Q_D = \pi R^2 \times 1.3\ C \times 5.7$

$= 23.6\ R^2\ C$

If the subgrade is loose and compressible

$N_c^1 = N_c$

Therefore $Q_D = 23.6\ R^2\ C$ for the worst case

The location of the load test which gave approximately the lowest subgrade bearing capacity on the Grande Prairie Airport will now be checked to obtain what load will be required to obtain a shear failure.

The location used is:

Runway - NE-SW

Station - 23 + 00
Offset - 33'L of centre line

---

* Dr. K. Terzaghi - Theoretical Soil Mechanics - Figure 38, Page 125

The triaxial tests from this location give the following:

Depth	0" - 6"	9" - 15"
C	1.3 kg/cm^2	0.55 kg/cm^2
ϕ	13.42°	29.0°

Therefore if one uses a $c = 0.55$ kg/cm^2 the worst case is being considered.

$$C = 0.55 \text{ kg/cm}^2 = 7.82 \text{ p.s.i.}$$

$$R = 15 \text{ inches}$$

Therefore $$Q_D = 23.6 \times 15 \times 15 \times 7.82 = 41{,}600 \text{ pounds}$$

The bearing capacity of the subgrade as found from the field loading test at this location was found to be less than 20,000 pounds. Therefore it appears from the theory that none of the failures were shear failures. Therefore it is reasonable to take a column of soil immediately below the plate and compute the compression of it.

2. Bearing Capacity Limited by Amount of Deflection of Subgrade Surface

Having assumed that the soil will not fail by shearing an attempt will now be made to correlate the bearing capacity of the subgrade as derived from the field tests with the results from certain laboratory tests as:

The Triaxial Compression Test
Moisture Content
Toughness
California Bearing Ratio

The thereoetical value of modulus of deformation of a soil as determined from an actual field test has been worked out.[*]

---

[*] Dr. A. Casagrande - in a discussion to paper by A.E. Cumming on "Distribution of Stresses Under a Foundation" - Proceedings - American Society of Civil Engineers October 1936 - Transactions No. 1936.

There are two assumptions or approximations.

1. The rate of volume decrease equal to the volume decrease observed in the consolidation test and is independent of the distortion of the mass.
2. The effect of the normal stresses on vertical planes, and of the shearing stresses on the volume decrease can be neglected.

A circular plate of diameter D, resting on the surface of a semi-infinite body with constant modulus of elasticity "E", is loaded with p/unit area. The theory of elasticity gives for the total displacement , Δ , of the centre of area (settlement) the following formula:

$$\Delta = \frac{p}{E} \times \frac{m^2 - 1}{m^2} \times D$$

in which "m" represents Poisson's *number.*

For perfectly incompressible material $m = 2$

For perfectly compressible material $m = \infty$

For $m = 2$ $\quad \Delta = \frac{3}{4}\frac{PD}{E}$

For $m = \infty$ $\quad \Delta = \frac{PD}{E}$

The settlement due to immediate deformation, without volume change, can be found by utilizing the stress strain diagram from an unconfined compression test on an undisturbed sample of clay. In the majority of cases the stress strain curve over range of stresses which are introduced into the soil by the load can be replaced by a straight line. The slope of this line is called modulus of deformation (M).

Since we have the incompressible case in this investigation,

$$\Delta = \frac{3pD}{4M}$$

The subsequent settlement, due to gradual volume decrease can be determined with help of the pressure-void ratio curve obtained from the consolidation test on an undisturbed sample. As a homogeneous mass is assumed the whole mass is preconsolidated under the same load, P_o , The loading raises this intrinsic stress in the soil in varying amounts, to a maximum of $(P_o + P)$. For this range of stress the change in volume may be represented by a straight line, the slope of which equals the coefficient of compressibility

$$a = \frac{e_1 - e_2}{P}$$

$$e = \text{void ratio}$$

From this the corresponding ratio of stress to strain is derived thus

$$A = \frac{P}{\dfrac{e_1 - e_2}{1 + e_1}} = \frac{1 + e_1}{a}$$

which is defined as the modulus of volume change.

This modulus corresponds to E for material obeying Hookes Law and with $m = \infty$

Therefore settlement of a circular area due to consolidation only can be found by replacing "E" with "A".

Hence the total settlement of the centre of this area will be

$$\Delta = \left(\frac{3}{2}\frac{1}{M} + \frac{2}{A}\right)pD$$

The settlement due to consolidation only

$$= \Delta = \frac{P}{A} D$$

Since in the field loading test as conducted in this investigation, the load was not on the soil long enough to produce consolidation, the case of load on an incompressible material is almost duplicated.

Therefore $\Delta = \frac{3\,p\,D}{4\,M}$

Since $D = 30''$

$p = \frac{P}{706}$ P = total load on plate

and $M = \frac{3\,p\,D}{4}$

From the field load test data the load P can be obtained for each value of Δ up to a $\frac{1}{2}$ inch. Therefore the corresponding values of "M" can be computed.

$M_{0.05}$ = Modulus of deformation for = 0.05 inches

$M_{0.1}$ = Modulus of deformation for = 0.1 inches

$M_{0.2}$ = Modulus of deformation for = 0.2 inches

As an example

suppose P = 7,200 pounds for = 0.05 inches

$p = \frac{7200}{706} = 10.20$ p.s.i.

Therefore $M_{0.05} = \frac{3 \times 10.20 \times 30}{4 \times 0.05} = 4600$ p.s.i.

$M_{0.05}$ and $M_{0.1}$ will now be computed for all Subgrade Load Tests conducted.

DATA FOR SUBGRADE LOAD TEST (30 Inch Plate) - GRANDE PRAIRIE

Station	Offset	P for $\Delta = .05$	P for $\Delta = .10$	P for $\Delta = 0.2$	P for $\Delta = 0.3$	P for $\Delta = 0.4$	P for $\Delta = 0.5$	M for $\Delta = 0.05$	M for $\Delta = 0.1$
14+00	33' L	4.97*	7.52*	10.78*	13.90*	16.88*	19.85*	2230	1692
	33' L	9.08	13.19	18.85	22.25	24.8	25.9	4040	2960
17+20	CL	11.33	15.60	20.7	24.8	28.2	30.9	5080	3820
18+00	33' L	12.05	16.15	22.0	26.9	30.6	33.8	5440	3640
	33' R	8.51	13.47	20.5	23.5	25.5	28.0	3840	3034
20+50	33' L	17.25	24.2	34.0	41.5	47.8	52.8	7760	5440
	33' R	10.62	18.85	30.0	37.2	42.3	46.3	4780	4240
23+00	33' L	10.20	15.88	25.1	32.9	39.7	44.8	4600	3580
	33' R	9.36	13.75	20.4	24.8	28.6	30.0	4220	3100
29+00	33' L	12.05	19.6	29.8	36.2	41.2	45.2	5440	4420
	33' R	10.92	17.03	25.5	30.5	34.3	37.4	2920	3840
35+00	33' L	18.00	24.4	35.6	45.0	52.5	59.5	8120	5480
	33' R	13.9	22.5	36.6	50.8	65.2	79.0	6260	5060
40+50	CL	7.06	22.2	33.2	41.8	49.0	54.7	3180	5020
45+00	33' L	12.32	18.6	29.5	38.3	45.5	52.0	5540	4180
	33' R	9.64	16.3	25.1	31.8	27.3	42.0	4340	3680

[illegible]

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DATA FOR SUBGRADE LOAD TEST (30 Inch Plate) - Grande Prairie

Station	Offset	P for $\Delta = 0.05$	P for $\Delta = .10$	P for $\Delta = 0.2$	P for $\Delta = 0.3$	P for $\Delta = 0.4$	P for $\Delta = 0.5$	M for $\Delta = 0.05$	M for $\Delta = 0.1$
5+50	25' R CL	8.80 *	15.5 *	24.1 *	30.5 *	35.9 *	40.6 *	3960	3490
8+80	35' L CL	12.90	18.8	27.3	34.3	40.0	44.7	5800	4240
	35' R CL	9.92	16.7	27.2	36.2	43.8	50.0	4460	3760
8+10	50' L EEPA	12.05	17.0	23.4	28.4	32.8	36.2	5420	3840
5+10	20' L	8.52	11.5	17.6	23.4	29.2	35.5	3840	2590
1+60	68' L	11.62	18.4	25.7	31.4	35.7	39.6	5240	4160
4+80	250' L	17.72	29.1	46.8	61.0	72.6	83.6	8000	6560
1+60	250' L	17.00	28.1	45.8	61.2	73.7	83.9	7660	6320
4+40	CL D.O.T.	13.74	21.6	33.8	43.6	51.3	58.4	6180	7860
3+70	180' R(LS)	16.90	27.1	42.8	56.0	66.6	75.2	7620	6100
2+70	60' R(LS)	6.67	8.08	7.06	17.0	18.8	20.6	3020	1800

DATA FOR SUBGRADE LOAD TEST (30 Inch Plate) - LETHBRIDGE

	Station	Offset	$P_{0.05}$	$P_{0.1}$	$P_{0.2}$	$P_{0.3}$	$P_{0.4}$	$P_{0.5}$	$M_{0.05}$	$M_{0.1}$	$M_{0.2}$
	4+00	CL	21.2	35.4	58.5	75.2	86.5	92.2	9540	7960	6580
	13+00	CL	17.3	27.2	41.2	51.1	59.5	66.7	7790	6120	4630
	20+00	CL	19.2	31.2	46.8	56.7	65.2	72.3	8640	7020	5260
	29+00	CL	18.4	30.9	51.1	66.7	80.8	92.2	8280	6950	5750
	7+00	CL	21.1	35.5	58.2	70.8	78.0	82.3	9500	8000	6550
	12+50	CL	21.4	32.5	46.8	58.2	66.7	70.8	9630	7320	5260
	22+50	CL	29.4	43.2	62.4	76.6	87.8	97.8	13,230	9720	7030
	31+00	CL	25.5	41.2	62.4	73.7	82.2	89.4	11,470	9280	7030
	5+00	CL	21.0	30.9	43.9	51.1	56.8	59.6	9450	6960	4940
SW an-strip	7+00	CL	24.0	38.0	59.5	75.1	89.2	99.3	10,800	8560	6700
	9+50	CL	19.8	29.1	43.9	53.8	62.4	68.1	8920	6550	4940
	4+00	CL	28.4	39.7	55.3	68.1	76.6	85.1	12,790	8930	6230
n	2+85	75' E	25.5	28.3	53.8	65.2	73.7	83.7	11,470	6380	6050
	5+47	112' E	16.4	25.5	36.4	44.2	51.2	57.0	8860	6890	4920 -36"
	8+00	37.5' E	22.1	36.3	61.0	82.2	102	119	9950	8060	6860
	12+25	40' E	17.0	28.8	48.2	62.3	75.1	83.7	7650	6480	5420
	7+00	37.5' E	21.4	36.3	59.5	72.3	77.9	80.8	9630	8280	6700
	9+00	115' E	16.3	34.0	46.8	55.2	62.3	67.0	7340	7660	5270

DATA FOR SUBGRADE LOAD TEST (30 Inch Plate) - STEVENSONS FIELD - WINNIPEG

	Station	Offset	$P_{0.05}$	$P_{0.1}$	$P_{0.2}$	$P_{0.3}$	$P_{0.4}$	$P_{0.5}$	$M_{0.05}$	$M_{0.1}$	$M_{0.2}$
5	3+04	CL	9.91*	15.00*	20.8*	24.8*	28.1*	30.3*	4460	3380	2360
	6+86	CL	9.35	14.30	20.6	25.5	28.5	32.6	4210	3220	2320
	10+86	CL	9.91	13.45	18.3	21.4	24.1	25.9	4460	3030	2060
	15+08	8' L	8.78	13.03	18.4	22.1	25.2	27.6	3950	2930	2070
	18+79	CL	10.62	16.60	24.2	28.3	31.3	33.3	4830	3730	2720
	23+04	10' R	9.21	13.05	17.7	20.6	22.7	24.6	4140	2940	1990
6	13+28	CL	9.50	14.18	18.6	21.2	23.0	24.1	4280	3190	2090
	21+28	CL	8.50	13.18	20.3	26.2	30.6	34.4	3830	2960	2280

DATA FOR SUBGRADE LOAD TEST (30 Inch Plate) - SASKATOON MAIN

Station	Offset	P for Δ =0.05	P for Δ =0.10	P for Δ =0.20	p for Δ =0.3	P for Δ =0.4	p for Δ =0.5	M for =0.05	M for Δ=0.1	M for Δ =0.2
21.+60	CL	7.87	11.48	17.6	22.3	27.2	31.5	3540	2580	1980
16+90	CL	8.51	11.32	17.2	20.6	23.4	25.8	3840	2540	1936
14+30	CL	11.33	15.75	24.8	30.2	34.0	37.4	5100	3540	2760
11+00	CL	9.50	15.04	22.0	26.2	29.4	31.8	4280	3380	2480
6+90	CL	9.22	13.90	20.7	25.6	29.2	32.0	4140	3140	2320
5+35	CL	8.37	12.32	18.0	22.4	22.9	24.4	3760	2780	2020
10+00	CL	4.26	6.52	11.3	15.9	20.6	25.1	1920	1408	1272
20+25	CL	7.09	11.34	20.0	28.6	37.2	45.3	3180	2560	2240
17+30	CL	6.24	10.07	15.9	20.3	23.7	26.2	2800	2400	1788
13+80	CL	6.52	9.93	16.9	22.7	28.0	33.6	2940	2240	1900
13+65	CL	9.22	14.90	23.4	30.0	35.3	39.7	4140	3360	2640
18+15	CL	8.51	12.48	17.7	21.3	24.1	26.4	3820	2800	1990
19+00	CL	7.08	10.78	17.0	21.7	25.7	28.9	3180	2420	1912
17+80	CL	10.63	16.45	25.4	32.6	38.6	44.0	4780	3700	2860
10+00	CL	9.22	15.06	23.8	30.2	35.2	39.0	4140	3400	2680
3+00	CL	5.68	8.80	13.1	16.3	18.7	20.3	2540	1564	1474
26+00	CL	7.37	12.07	19.2	24.2	28.4	31.6	3320	2720	2160
2+50	CL	9.45	14.18	20.6	23.2	27.1	28.5	4240	3160	2320

There should be a linear relationship between $M_{0.05}$, $M_{0.1}$, $M_{0.2}$ etc. The relationship should be better between $M_{0.05}$ and $M_{0.1}$ as lower loads were applied to the plates for these values. Therefore many of the variables that come in at the higher values of deflection and load will not effect the Modulus of Deformation so much in the lower ranges of load and deflection. The author will therefore only use $M_{0.05}$ and $M_{0.1}$ as obtained from the field loading tests.

A plot of "$M_{0.1}$" vs "$M_{0.05}$" (Figure 10) for the field loading tests conducted on the Grande Prairie Airport shows that this straight line relationship actually holds and therefore it doesn't make much difference whether $M_{0.05}$ or $M_{0.1}$ is used. $M_{0.05}$ will probably be the more reliable.

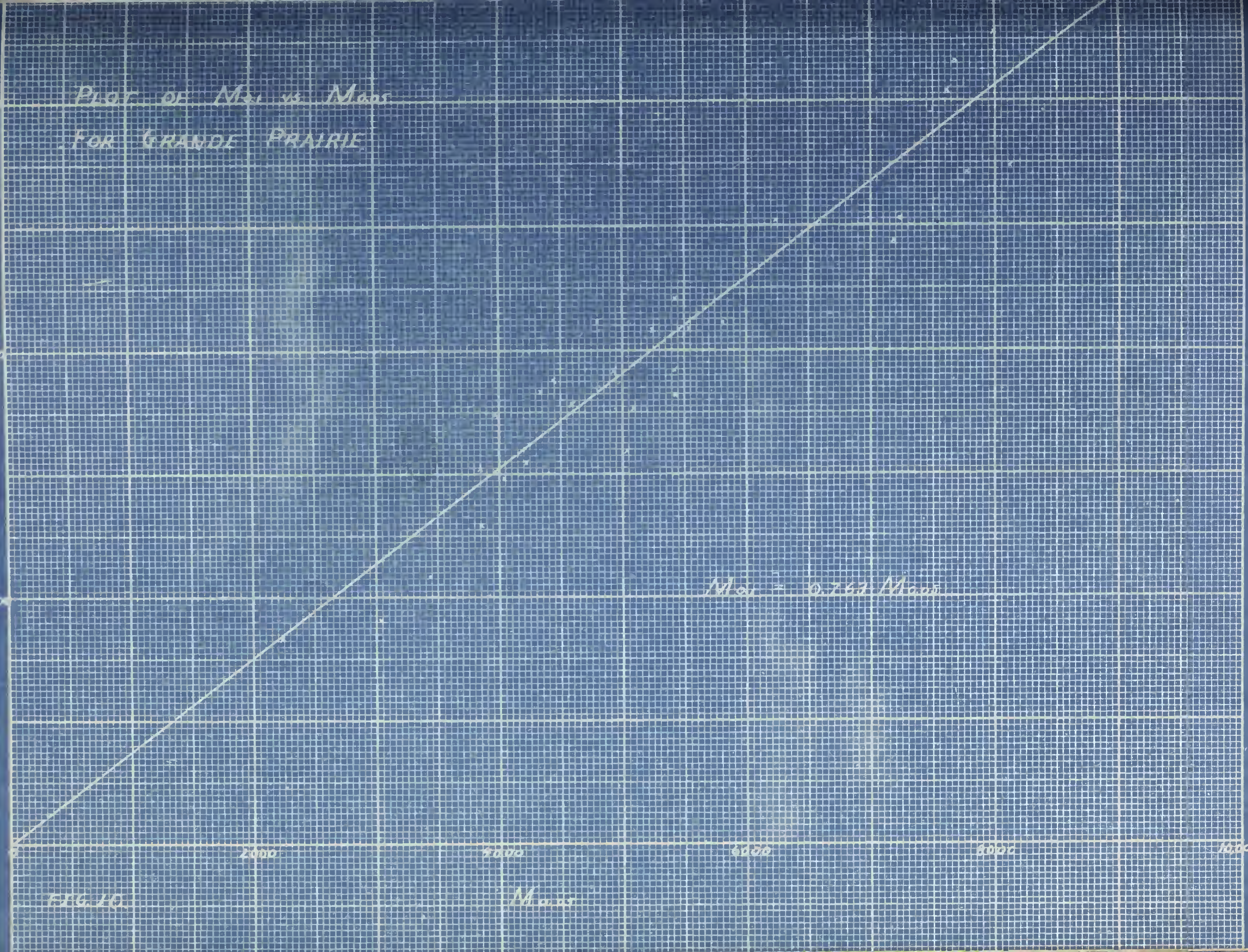

Plot of Mci vs Mass
For Grande Prairie
Mci = 0.763 Mass
2000
4000
6000
8000
10,000
Mass

FIG. 10

From this work it appears that there should be a linear relationship between the modulus of deformation obtained from the field test and that obtained from the triaxial test.

Triaxial data is available for most locations for depth of subgrade of 0"-6" and of 9"-15". An examination of this data shows that there is a wide discrepancy in the modulus of deformation for the soils from different depths at the same location. The problem now rises as to how one should apportion the differences in modulus of deformation.

It does not seem reasonable to average the two values of "M" as the sample which has the lower value of M will be the softer material and therefore this soil in place would produce more deflection for lower values of stress. This problem will therefore be attacked from a stress-strain view point.

The vertical normal stress beneath the centre of a uniformly loaded circular area has been published in table form.*

The vertical normal stress at a depth of "z" beneath the centre of a circular area with radius R carrying a load p per unit of area is

$$\sigma_z = p I_\sigma$$

wherein $$I_\sigma = 1 - \left[\frac{1}{1 + (R/z)^2}\right]^{3/2}$$

The table gives values of I_σ for different values of R/z.

As the samples used in the laboratory triaxial tests were taken from the field from z = 0 to 6" and 9"-15", the stress at z = 3" and z = 12" will be computed vertically below the centre of the plate.

At z = 3 $\quad R/z = \frac{15}{3} = 5$

$I_\sigma = 0.99246$

$\sigma_z = 0.99246\ P$

At z = 12 $\quad R/z = \frac{15}{12} = 1.25$

$I_\sigma = 0.75622$

$\sigma_z = 0.75622\ p$

---

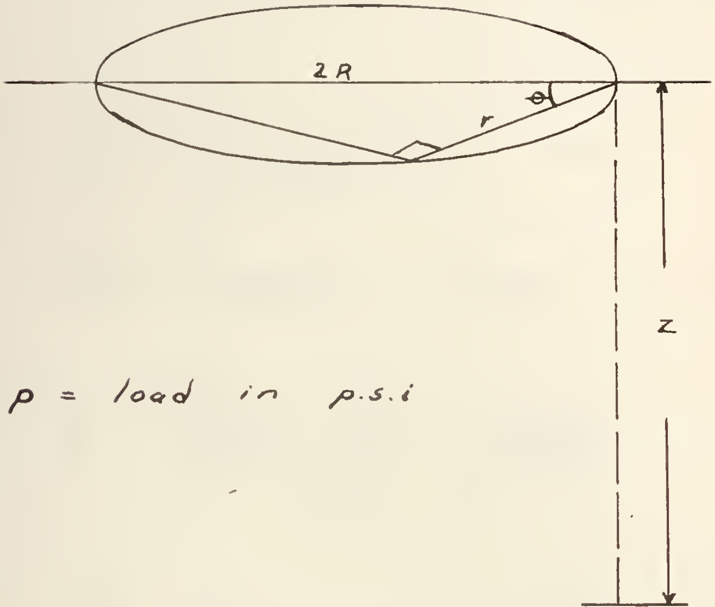
* Dr. K. Terzaghi -
Theoretical Soil Mechanics
Appendix - Table III.

To find the vertical normal stress beneath the edge of a uniformly loaded circular area is much more difficult.

Bousinesques equation for the state of stress in a homogeneous mass, due to a surface load, has been derived in the following form.*

$$\sigma_z = \frac{3P}{2\pi} \frac{z^3}{(r^2+z^2)^{5/2}} \, dA$$

The following solution can be worked out for the stress vertically below a uniformly loaded plate.



p = load in p.s.i

$$\sigma_z = \frac{3p}{2\pi} \frac{z^3}{(r^2+z^2)^{5/2}} \, r \, d\theta \, dr$$

$$= \frac{3pz^3}{2\pi} \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} \int_{0}^{2R\cos\theta} \frac{r}{(r^2+z^2)^{5/2}} \, d\theta \, dr$$

$$= \frac{3pz^3}{2\pi} \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} \left[\frac{(r^2+z^2)^{-3/2}}{-3} \right]_0^{2R\cos\theta} d\theta$$

---

* S. Timoshenko - Theory of Elasticity

$$\sigma_z = \frac{p z^3}{2\pi} \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} \left[\frac{1}{z^3} - \frac{1}{(4R^2\cos^2\theta + z^2)^{3/2}} \right] d\theta$$

$$= \frac{p}{2} - \frac{z^3 p}{2\pi} \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} \frac{d\theta}{(4R^2\cos^2\theta + z^2)^{3/2}}$$

$$= \frac{p}{2} - \frac{z^3 p}{\pi(4R^2+z^2)^{3/2}} \int_0^{\frac{\pi}{2}} \frac{d\theta}{(4R^2 + z^2 - 4R^2\sin^2\theta)^{3/2}}$$

$$= \frac{p}{2} - \frac{z^3 p}{\pi(4R^2+z^2)^{3/2}} \int_0^{\frac{\pi}{2}} \frac{d\theta}{(1 - k^2\sin^2\theta)^{3/2}}$$

$$k^2 = \frac{4R^2}{4R^2+z^2} < 1$$

From Haan's Tables of Integrals.

$$\sigma_z = \frac{p}{2} - \frac{z^3 p}{\pi(4R^2+z^2)^{3/2}} \frac{E(k)}{1-k^2}$$

$$= \frac{p}{2} - \frac{z^3 p}{\pi(4R^2+z^2)^{3/2}} \frac{E(k)}{1 - \frac{4R^2}{4R^2+z^2}}$$

$$= \frac{p}{2} - \frac{z\,p\;E(k)}{\pi\sqrt{4R^2+z^2}}$$

$$E(k) = \int_0^{\frac{\pi}{2}} \sqrt{1 - k^2\sin^2\theta}\; d\theta.$$

Very near the surface (i.e. when z →0) the stress immediately below the edge of the plate equals ½P. Just in from the edge the stress equls P. Stress just outside the edge equals zero. This discrepancy rapidly disappears as z increases.

Now the stresses beneath the centre of the plate are:

at $z = 3$ $\quad \sigma_z = 0.992\ P$

at $z = 12$ $\quad \sigma_z = 0.756\ P$

From the above work the stress below the edge of the plate can be computed

$$\sigma_z = \frac{P}{2} - \frac{zP}{\pi\sqrt{4R^2+z^2}} \quad E(k)$$

$$k = \sqrt{\frac{4R^2}{4R^2+z^2}} \qquad R = 15 \text{ inches.}$$

$$k = \sqrt{\frac{900}{900+z^2}}$$

$$\therefore \sigma_z = \frac{P}{2} - \frac{zP}{\pi\sqrt{900+z^2}} \quad E(k)$$

At $z = 3''$ $\quad k = \sqrt{\frac{900}{909}} = 0.995$

$$\text{Stress} = \frac{P}{2} - \frac{3P}{\pi\sqrt{909}} \quad E(k)$$

$$= \frac{P}{2} - \frac{3P}{94.7} \quad E(0.995)$$

$$= \frac{P}{2} - 0.034\ P$$

$$= \underline{0.466\ P.}$$

At $z = 12''$ $\quad k = \sqrt{\frac{900}{1044}} = 0.928$

$$E(k) = 1.136$$

$$\text{Stress} = \frac{P}{2} - \frac{12P}{\pi\sqrt{1044}} \times 1.136$$

$$= \underline{0.366\ P}$$

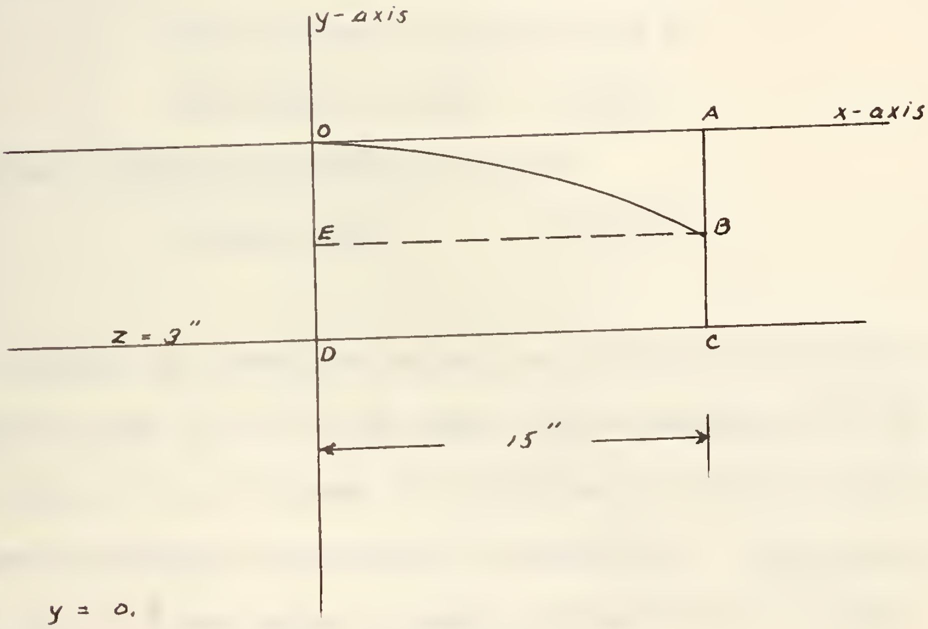
At z = 3" and z = 12" it will be assumed that the distribution diagram of stress on a horizontal plane is of the form of a parabaloid. Using this assumption one can obtain a more accurate value for average stress on the plane in question than by assuming a linear relationship. Most of the literature also indicates that the stress distribution diagram is parabolic.

In this work the value for average stress is the average stress on a horizontal plane in a vertical column, of diameter equal the diameter of the plate, below the plate.

The stress distribution curve will be assumed to be a parabola the equation of which is $y = c.x^2$

at z = 3 σ_z under centre of plate = 0.992 P

σ_z under edge of plate = 0.466 P



when

x = 0 , y = 0.

x = 15 , y = 0.466.-0.992.

Assuming the axis as shown in the figure, when $x = 15''$

$$y = .992 - .466 = 0.526$$

Therefore $0.526 = C_1 \times 15^2$

$$C_1 = \frac{.526}{225} = .00233$$

Therefore

$$y = .00233\, x^2$$

$$\text{Area OAB} = \int_0^{15} 0.00233\, x^2\, dx$$

$$= \left[0.00233\,\frac{x^3}{3}\right]_0^{15} = 2.62$$

$$\text{Area ODAC} = .992 \times 15 = 14.85$$

$$\text{Therefore Area ODBC} = 14.88 - 2.62 = 12.26$$

Therefore Average

$$\text{Stress} = \frac{12.26}{15} = 0.817$$

$$= 0.817\ P$$

at $z = 12''$

under centre of plate $= 0.756\ P$

under edge of plate $= 0.366\ P$

Assuming parabolic distribution as above

Average stress $= 0.626\ P$

Now to obtain the average value for the "Modulus of Deformation" obtained from laboratory tests run on the cylinders taken from depths 0-6 and 9-15 inches below the surface of the subgrade, the difference in the values for "M" shall be apportioned according to unit strain at each depth. The strains shall be computed by using the theoretical stresses obtained at $z = 3$ inches, and $z = 12$ inches and the modulus of deformation obtained from the triaxial tests in the laboratory for the respective depths.

The theory for the calculation of these stresses is based on the assumption that a homogeneous isotropic soil is being dealt with. This is not the case with the large variations in moisture content that are obtained in the field, and if the soil was homogeneous and isotropic the values of "M" from the Triaxial Test for the two depths would be identical.

The value of "M" from the field test is computed from results derived from the field loading test; for all practical purposes the plate had finished settling when the results that were used were recorded. It is therefore reasonable that the soil can be assumed to be homogeneous and isotropic, at this time, as far as distribution of stress is concerned.

The following is a typical example showing the manner in which the laboratory values of Modulus of Deformation were averaged.

Grande Prairie

Runway - NW-SE #11

Station - 17 + 20

Offset - 0

Datum - CL

Data from Triaxial Test

LATERAL PRESSURE	M		
	0 - 6"	9 - 15"	Averaged
0 p.s.i.	71	97	80.0 kg/cm^2 = 1138 p.s.i.
15	185	161	173.8 kg/cm^2 = 2470 p.s.i.
30	240	192	216.6 kg/cm^2 = 3080 p.s.i.

For lateral pressure = 0 p.s.i.

Unit strain at z = 3 = $\frac{.817 P}{71} = .0115 P$

at z = 12 = $\frac{.626 P}{97} = .00646 P$

Therefore "M" (averaged) = $71 + \frac{.00646}{.01796}(97-71) = 80 \text{ kg/cm}^2$

For lateral pressure = 15 p.s.i.

Unit strain at z = 3 = $\frac{.817 P}{185} = .00340 P$

at z = 12 = $\frac{.626 P}{161} = .00326 P$

Therefore "M" (averaged) = $185 - \frac{.00326}{.00813}(185-161) = 173.8 \text{ kg/cm}^2$

For lateral pressure = 30 p.s.i.

Unit strain at z = 3 = $\frac{.817 P}{240} = .00340$

at z = 12 = $\frac{.626 P}{192} = .00326$

Therefore "M" (averaged) = $240 - \frac{.00326}{.00666}(240-192) = 216.6 \text{ kg/cm}^2$

To convert these averaged values of "M" from kg/cm^2 to p.s.i. multiply by 14.22

The term average as used above does not mean average in the true sense of the word and is only used for lack of a better term.

Now a plot of "M" (averaged) from the triaxial tests and "$M_{0.05}$" and "$M_{0.1}$" can be made on arithmetic paper.

Triaxial Data from Soils in Zone C

Runway	Station	Offset	Lat. Press.	M kg/cm^2		C		Averaged M p.s.i.	Moisture Content	
				0-6	9-15	0-6	9-15		0-6	9-15
#11 NW-SE	17+20	CL	0	71	97			1138	25.9	31.0
			15	185	161	.85		2470	26.2	31.3
			30	240	192			3080	24.6	31.2
# 11 NW-SE	23+00	33' L	0	110	163			1820		13.0
			15	151	-	.55	1.30	1950	13.3	16.3
			30	126	196			2120	14.4	
#11 NW-SE	29+00	33' R	0	114	98			1510	21.2	16.3
			15	161	176	.62		2220	24.5	17.9
			30	206	204			2910	24.2	17.6
T.S. #1	2+70	60'R-LS	0	76	31.8			677	30.2	32.6
			15	125	30.7	.35		1220	31.0	31.9
			30	145	75.3			1770	31.2	32.0

Triaxial Data from Soils in Zone D

			Lat.	M kg/cm^2		C		Averaged M	Moisture Content	
Runway	Station	Offset	Press.	0-6	9-15	0-6	9-15	p.s.i.	0-6	9-15
#26 EW	13+ 20	CL	0	50	61			767	33.3	40.3
		CL	15	64	105	.23	.15	1090	32.7	40.3
			30	111	160			1820	33.7	38.8
#26	21+20	CL	0	68	58			900	33.2	37.8
			15	124	107	.15	.25	1650	32.6	39.8
			30	288	306			4200	32.7	39.1
#35 NS	23+14	CL	0	113	60			853	28	22.1
			15		75		.5	1060		30.1
			30		135			1920		
#35	19+07	CL	0	104	50			1010	19.3	22.8
			15	134	85	.35	.5	1520	25.6	25.7
			30		150			2370		21.6
	3+27	CL	0	67	61.5			916	29.5	38.0
			15	100	81.7	.70		1330	28.1	29.0
			30	145	106			1770	28.8	44.7
	6+70	CL	0	53				755	30.8	
			15	98		.45		1390	32.0	
			30	135				1920	30.3	
	11+08	CL	0		14.3			203		34.2
			15		53.0			755		34.8
			30		85.0			1210		33.3
	15+08		0	55	53			768	23.9	23.3
			15	90	78	.5	.55	1200	22.2	24.1
			30	150	156			2160	26.2	22.8

Triaxial Data from Soil in Zone A

Runway	Station	Offset	Lat. Press.	M kg/cm^2		C		Averaged M	Moisture Content	
				0-6	9-15	0-6	9-15	p.s.i.	0-6	9-15
NW-SE	13/00	15' W	0	120				1710	14.3	
			15	200				2850	14.2	
			30	228				3240	14.3	
	20/00	CL	0							
			15							
			30							
	29/00	CL	0		75			1070		10.5
			15		110			1570		9.6
			30		240			3410		9.1
NE-SW Taxi Strip	5/00	4' N	0		90			1270		19.7
			15		115			1640		17.9
			30		390			5550		16.9
NE-SW	7/10	CL	0	162	90			1710	13.8	12.1
			15	246	110	.63	.40	2270	14.5	15.0
			30	301	232			3800	13.9	12.3
	12/55	CL	0	162	125			2050		13.6
			15	246	200	.64		3180		13.6
			30	301	350			4550		12.8
	22/05	CL	0	99	93			1360	7.9	9.3
			15	100	205	.3	.65	1820	8.7	9.6
	30/95	CL	0	130				1850	9.3	
			30	290				4120	11.0	
Connecting Strip NE SW & EW	9/50	CL	0	100	119			1520	14.4	17.6
			15	254	145			2720	15.8	16.6
			30		202					13.3

Triaxial Data from Soils in Zone A

Runway	Station	Offset	Lat. Press.	M kg/cm^2		C		Average M p.s.i.	Moisture Content	
				0-6	9-15	0-6	9-15		0-6	9-15
Con. Strip NE-SW & EW Runway	9+45	CL	0	100	119			1240	14.4	17.6
			15	254	145	.95	.50	1720	15.8	16.6
			30		202			3800		13.3
N.S. Apron	3+00	CL	0	85	90				17.5	17.5
			15	122	120	1.0	.25		17.3	10.9
			25	277	250				17.1	13.1
NW-SE	4+00	CL	0	200	150			2490	10.9	10.2
			15	278	150		.95	2890	11.2	10.1
			30	438	111			4980	12.2	10.7
N.S. Apron	5+47	112' E of W Apron	0	135	210			2280	13.0	14.2
			15	220	337	.75	.70	3680	13.3	14.6
			30	280	184 ?				13.2	13.7
N.S. Apron	7+05	37½' E of W Apron	0		71			1010		17.8
			15		84		.50	1190		19.3
			30		97			1380		14.0
Con.Strip NE-SW & EW Runway	3+95	CL	0	195	255			3080	11.3	7.8
			15	220	290	.52	.50	3500	11.0	8.6
			30	446	121 ?			4120	12.3	8.5
Con. Strip NE-SW & EW Runway	7+05	CL	0		80			1210		13.5
			15	137	125	1.0		1880	15.3	12.6
			30	150	254			2680	15.4	13.6
Con. Strip	12+30	40' E	0	105	180			1820	9.2	7.7
			15	125	110	.58	.40	2000	9.0	7.8
			30	125	300				8.7	8.7
N.S. Apron	8+95	115 ' E	0		75			1070		16.0
			15		105			1500		16.5
			30		250			3560		16.0

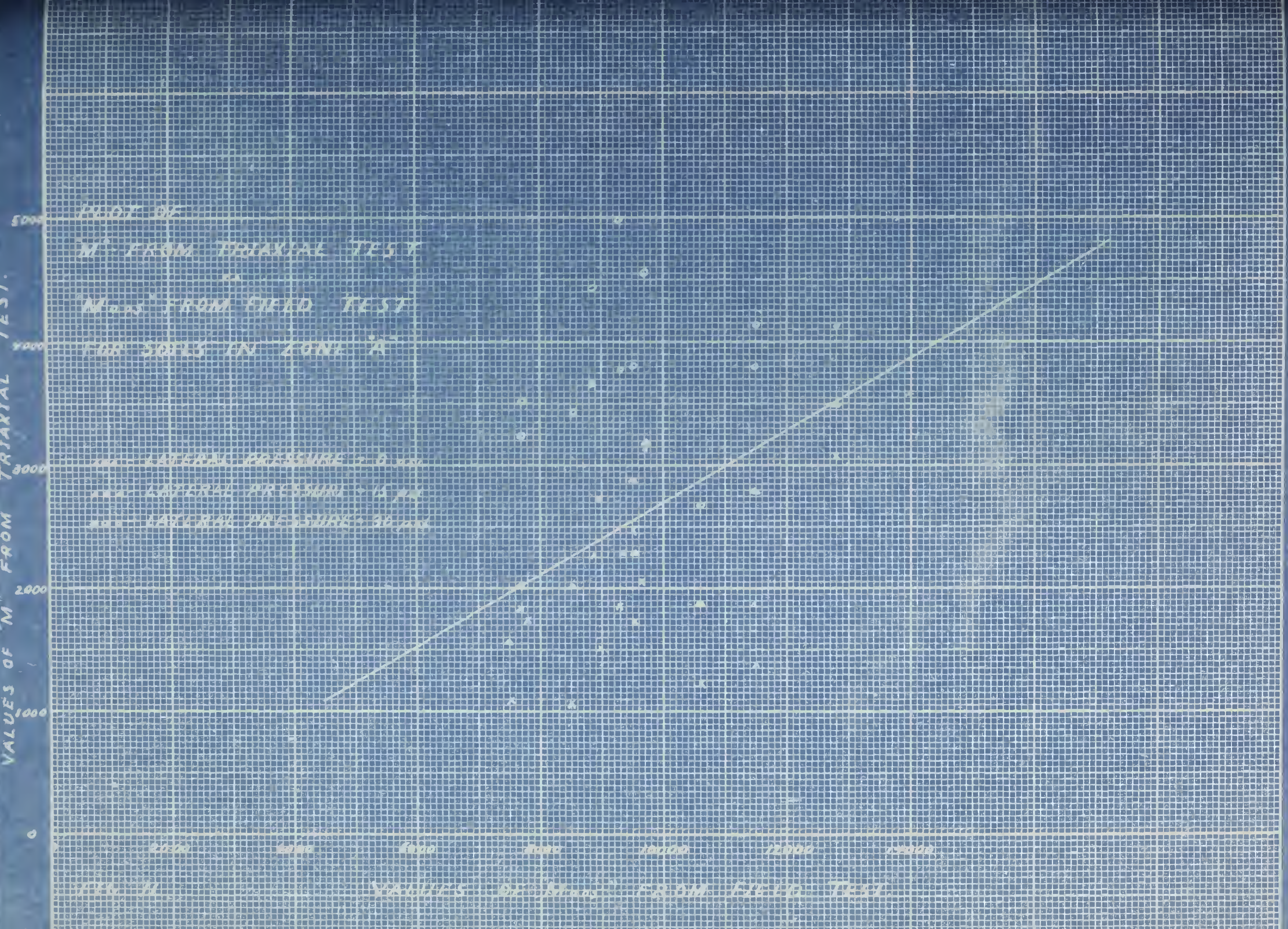

PLOT OF
M FROM TRIAXIAL TEST
&
$M_{0.5}$ FROM FIELD TEST
FOR SOILS IN ZONE A
— LATERAL PRESSURE = 0 PSI
— LATERAL PRESSURE = 15 PSI
— LATERAL PRESSURE = 30 PSI
VALUES OF "M" FROM TRIAXIAL TEST.
0
1000
2000
3000
4000
5000
2000
4000
6000
8000
10000
12000
14000
VALUES OF $M_{0.5}$ FROM FIELD TEST

FIG. 11

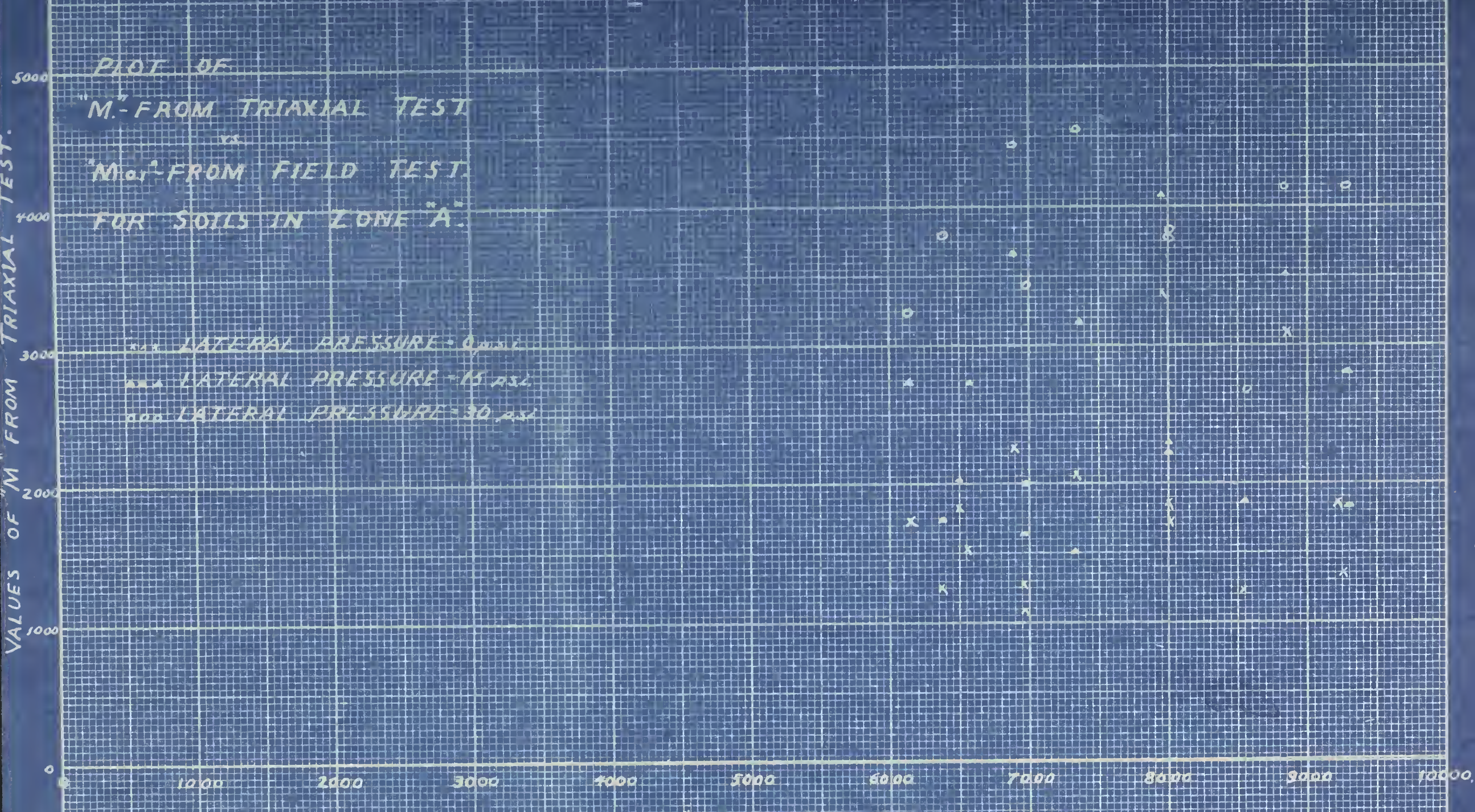

PLOT OF
"M." FROM TRIAXIAL TEST
vs
"Mat" FROM FIELD TEST
FOR SOILS IN ZONE "A".
x x x LATERAL PRESSURE - 0 psi.
▲▲▲ LATERAL PRESSURE - 15 psi.
o o o LATERAL PRESSURE - 30 psi.
VALUES OF "M" FROM TRIAXIAL TEST.
0
1000
2000
3000
4000
5000
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000.
VALUES OF "Mat" FROM FIELD TEST.

FIG 12.

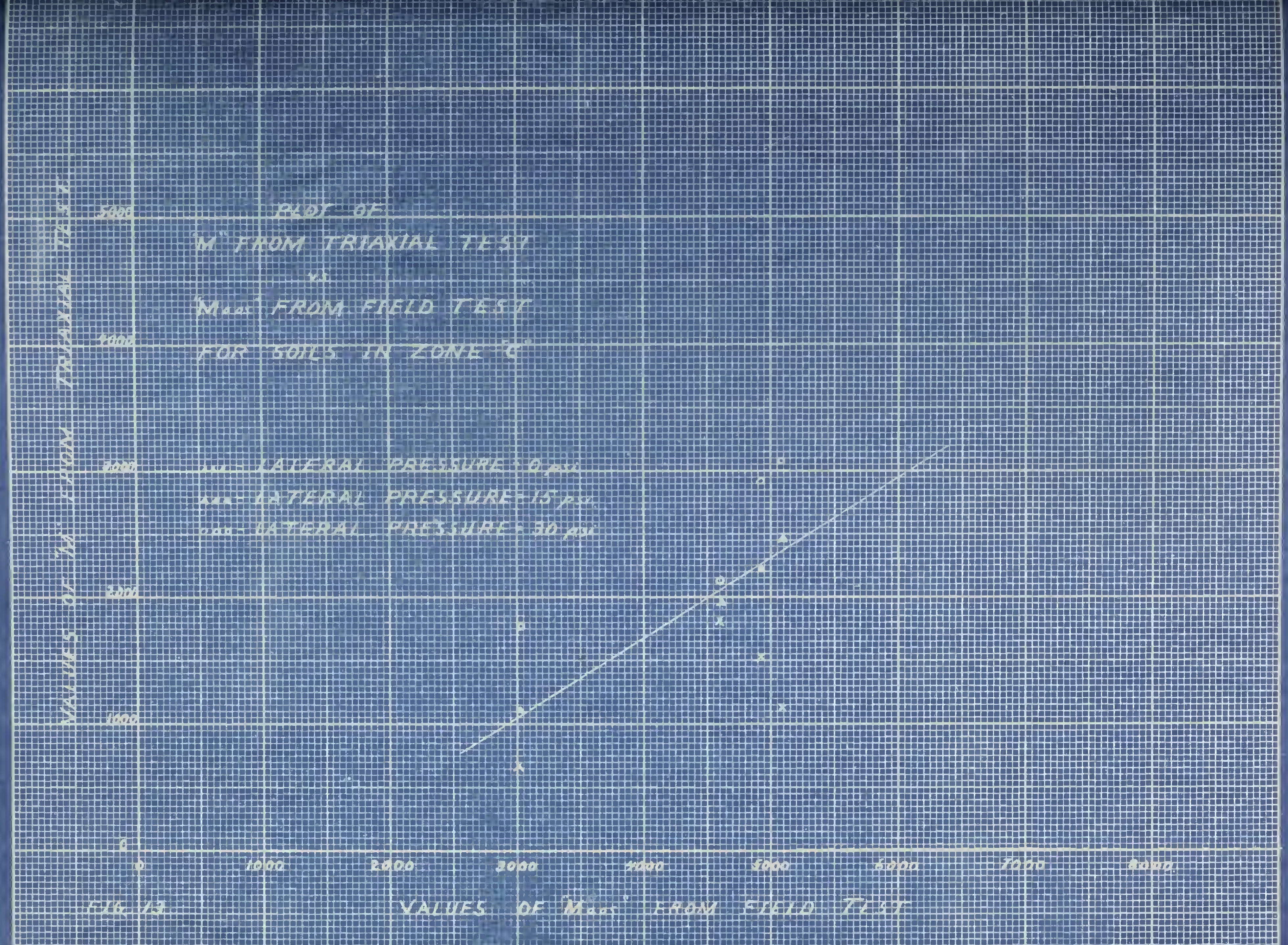


FIG. 13

PLOT OF
"M" FROM TRIAXIAL TEST
vs
"M[illegible]" FROM FIELD TEST
FOR SOILS IN ZONE "C"

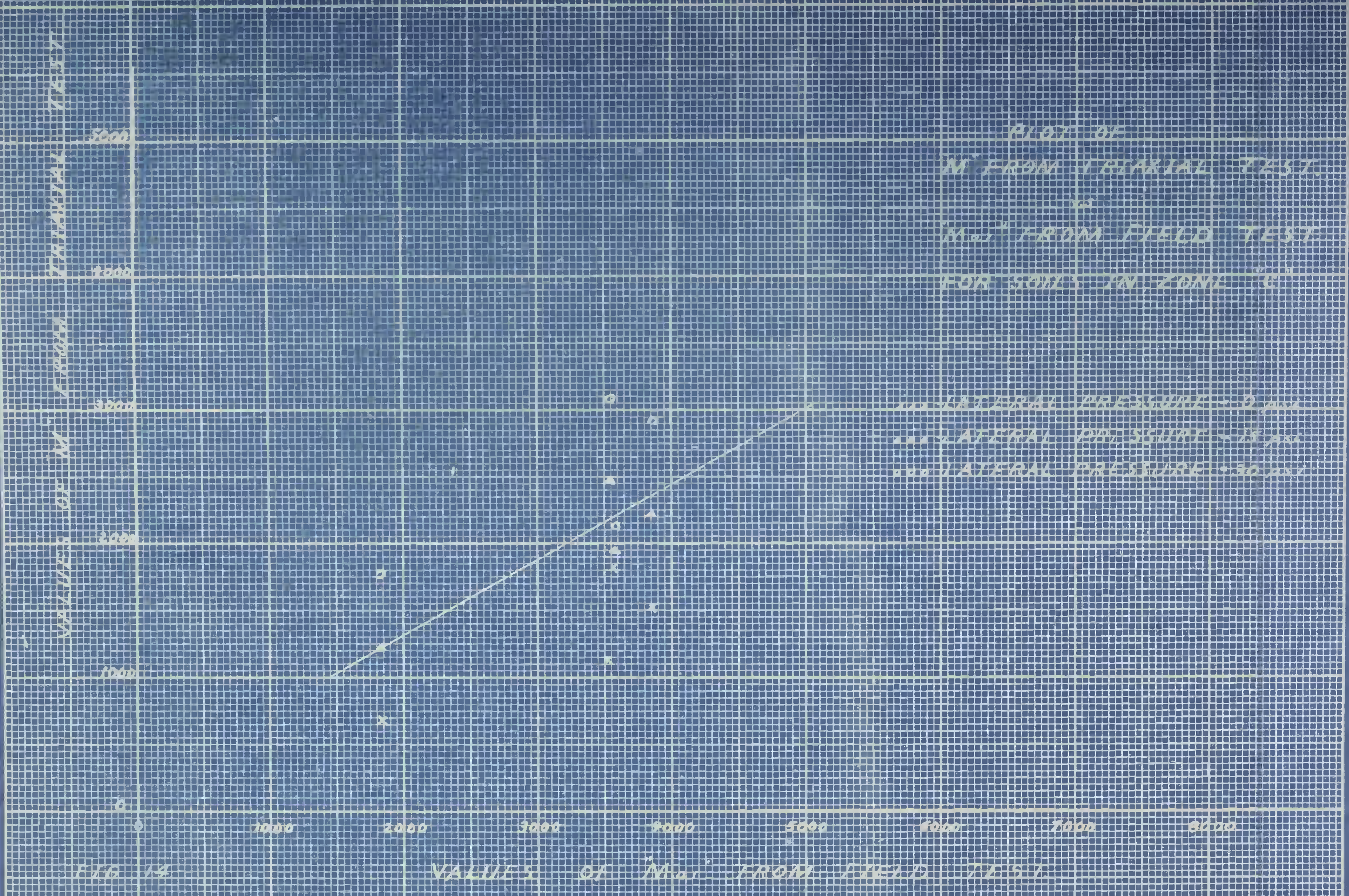

PLOT OF
M" FROM TRIAXIAL TEST.
vs
"Mo" FROM FIELD TEST
FOR SOIL IN ZONE "C"
LATERAL PRESSURE = 0 psi
LATERAL PRESSURE = 15 psi
LATERAL PRESSURE = 30 psi
VALUES OF M FROM TRIAXIAL TEST
0
1000
2000
3000
4000
5000
0
1000
2000
3000
4000
5000
6000
7000
8000
VALUES OF "Mo" FROM FIELD TEST

FIG 14

From the plots of "M" from the Triaxial Test vs "M" from the field tests (Figures 11-14) it can easily be observed that the best results are obtained for "M" from triaxial test (Lateral pressure = 15 p.s.i.) vs "$M_{0.05}$" from the field test (i.e. "M" for deflection of o.05 inches).

Therefore the only plots that will be made henceforth will be "M" (lateral pressure = 15 p.s.i.) from triaxial test vs "$M_{0.05}$" from field tests. To avoid confusion a plot of the curves for zone A and C will be repeated plotting "M" from triaxial test (lateral pressure = 15 p.s.i.) vs $M_{0.05}$ from the field test.

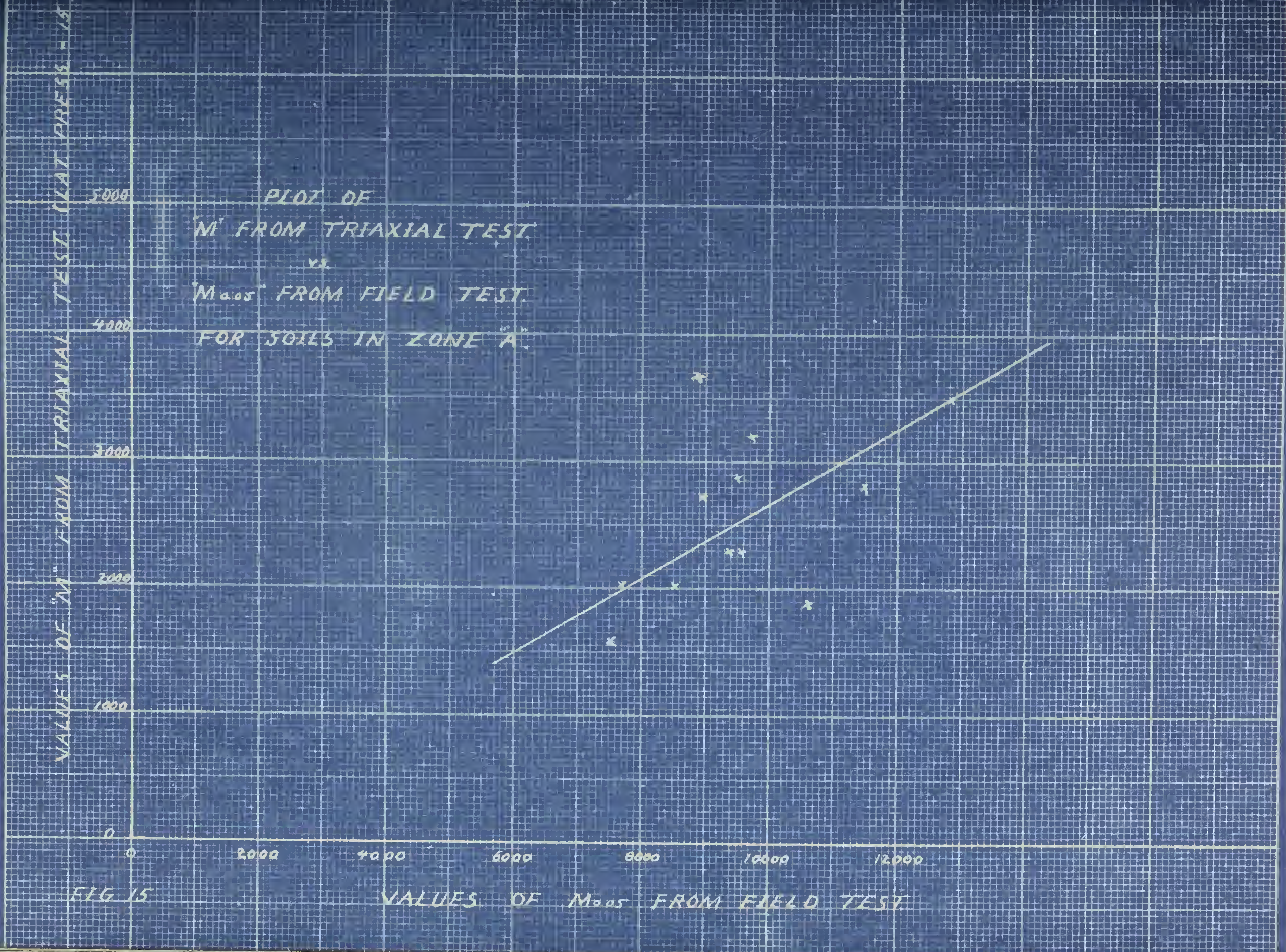

PLOT OF
"M" FROM TRIAXIAL TEST
vs
"M₀.₀₅" FROM FIELD TEST
FOR SOILS IN ZONE "A"
VALUES OF "M" FROM TRIAXIAL TEST (LAT PRESS - 15
0
1000
2000
3000
4000
5000
0
2000
4000
6000
8000
10000
12000
VALUES OF M₀.₀₅ FROM FIELD TEST

FIG 15

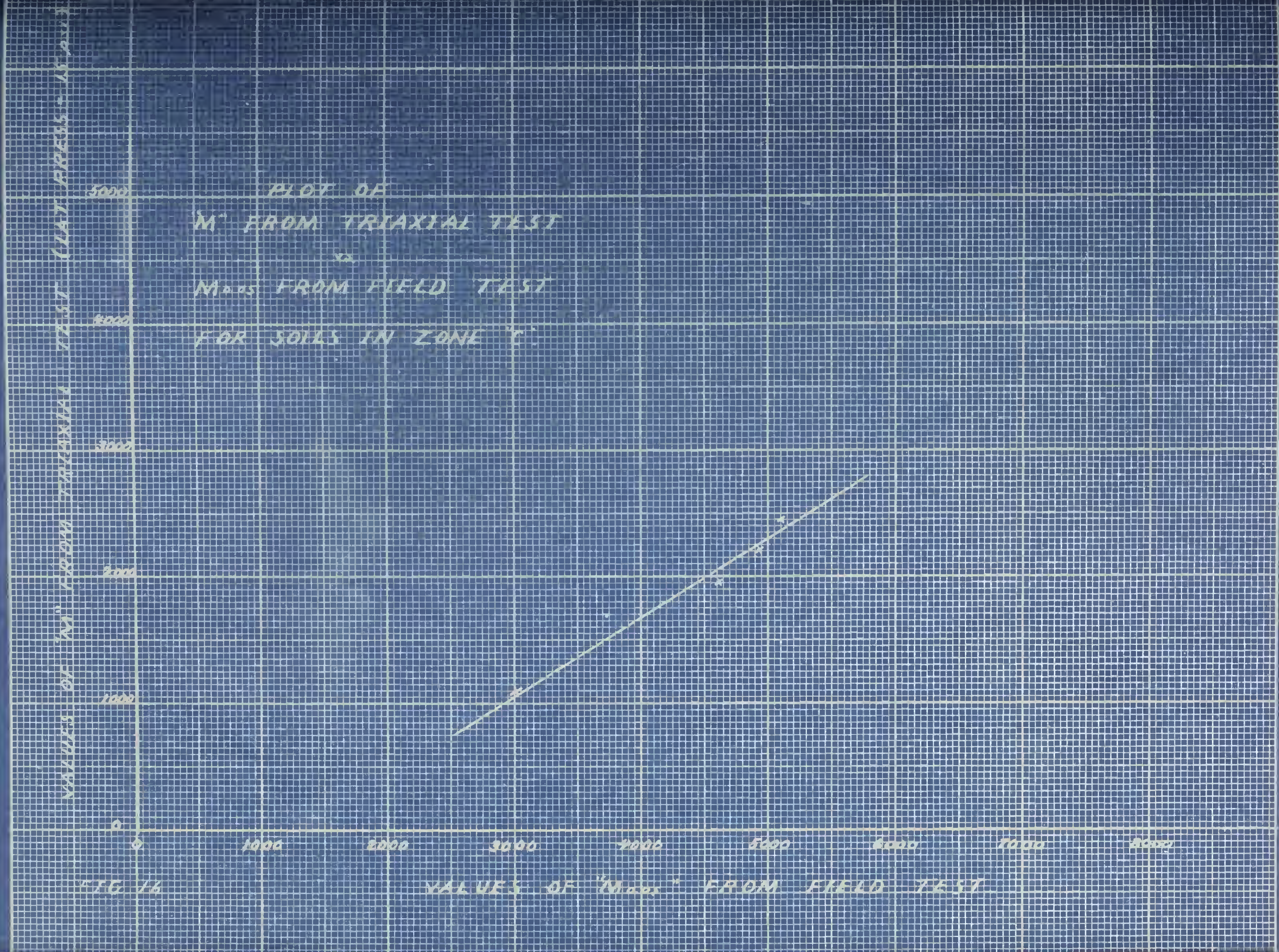
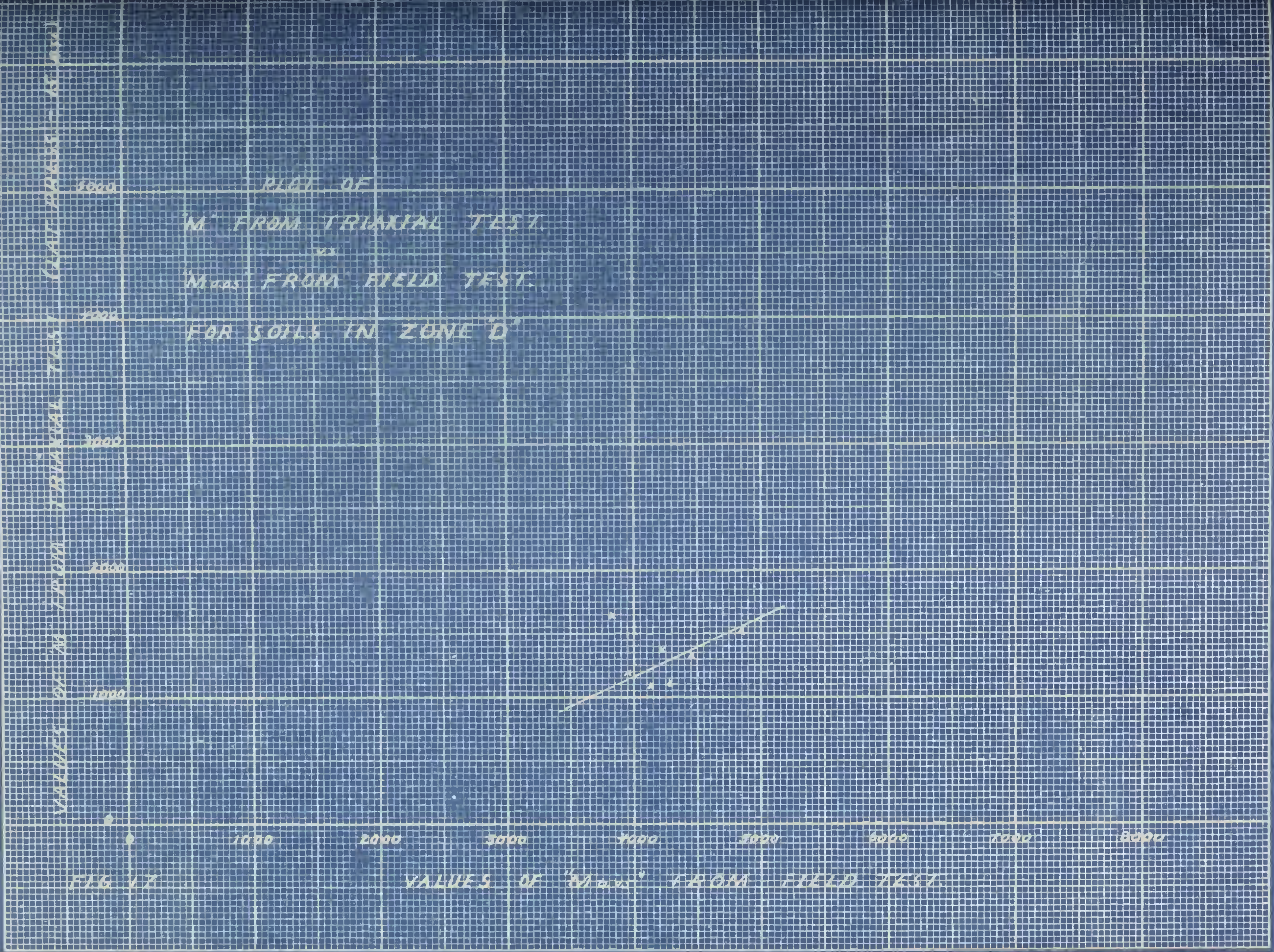

PLOT OF
M' FROM TRIAXIAL TEST
to
Mobs FROM FIELD TEST
FOR SOILS IN ZONE 'I'
VALUES OF "M" FROM TRIAXIAL TEST (LAT PRESS = 15 p.s.
VALUES OF "Mobs" FROM FIELD TEST
0
1000
2000
3000
4000
5000
6000
7000
8000
0
1000
2000
3000
4000
5000

FIG 16



PLOT OF

M FROM TRIAXIAL TEST

vs

Mess FROM FIELD TEST

FOR SOILS IN ZONE "D"

FIG. 17 VALUES OF "Mess" FROM FIELD TEST

An examination of these charts (Figures 15-17) shows that with the exception of 2 points on the plot for zone "A" and one point on the plot for Zone "D", a fairly good straight line relationship holds.

The results from the subgrade load tests on the soils in zone D were all very nearly the same and therefore the points on the plot for that zone are grouped fairly close together. Even so a straight line relationship seems to exist.

The high value of the one point on the plot for soils in zone "A" can be explained by the fact that samples in the laboratory had 2% less moisture at the time the triaxial tests were run than did the soil as tested in the field.

Other variations from the straight line relationship could probably be accounted for by one or more of the following lines of reasoning:

1. Some of the triaxial samples dried out somewhat before they were tested.
2. Some of the samples were stored in a wet room where water was allowed to drip onto them. A small leak in the seal would cause soaking up of water by the sample.
3. In the field there may have been a soft or hard stratum below the section that was sampled. A soft stratum would likely have more effect than a hard stratum.
4. Rate of loading triaxial samples in the laboratory seems to have considerable effect on values for modulus of deformation.

Now with the use of these charts and the classification charts, a fairly reliable value for the strength of a subgrade material can be determined.

3. California Bearing Ratio Method

The prevailing method of design of airport runways by the United States Engineering Department is the California Bearing Ratio Method (C.B.R.). Therefore an effort will now be made to correlate the C.B.R. data with the field load tests and the classification tests.

The C.B.R. test in the laboratory was run first on the sample in the undisturbed state and then on the same sample saturated.

If the C.B.R. method of design is good then there should be some relation between the C.B.R. values as obtained from the sample in the natural state and the values of "M" as derived from the field load tests.

A plot therefore of C.B.R. (natural sample) vs $M_{0.05}$(from Field Tests) is shown in Figure 18.

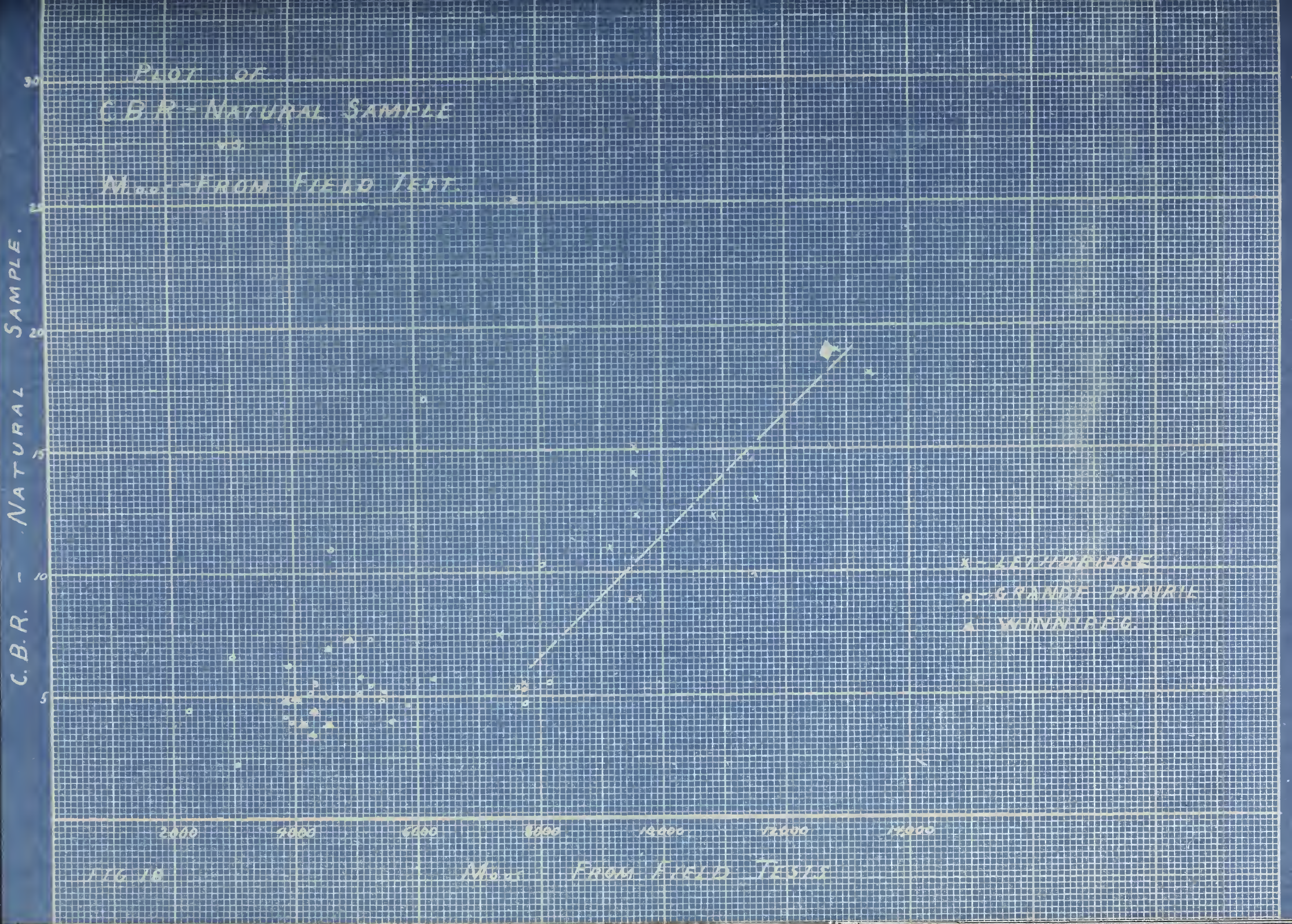

PLOT OF
C.B.R.-NATURAL SAMPLE
vs
MOD.-FROM FIELD TEST.
C.B.R. - NATURAL SAMPLE.
30
25
20
15
10
5
2000
4000
6000
8000
10000
12000
14000
MOD. FROM FIELD TESTS
x - LETHBRIDGE
o - GRANDE PRAIRIE
▲ - WINNIPEG

FIG. 18

The values of C.B.R. for Winnipeg and Grande Prairie were rather low, between 3 and 8. They are, however, for the most part fairly well grouped in one location on the graph.

The C.B.R. values for Lethbridge ranged between 5 and 20 and it was found that a linear relationship appeared to exist as shown by the dashed line on the graph.

This plot indicates that the C.B.R. could be used for the design of airport runways if a better method could not be found. There is, however, quite a large discrepancy in the values one can obtain for $M_{0.05}$ from this plot for one individual value of C.B.R.

From this work it appears that if this method of design was used one should zone the classification charts as has been suggested before, and have a different set of design curves for each zone. The plot of C.B.R. vs $M_{0.05}$ verifies this somewhat. Lethbridge would be in one zone and Winnipeg and Grande Prairie in another. It is easily seen that for the same value of C.B.R. the Lethbridge subgrade is stronger than either the Winnipeg or Grande Prairie subgrade.

The classification charts gave a linear relationship between plasticity index and liquid limit. Dr. Terzaghi claims that the soils at the lower end of this line are coarser grained than those at the other end. Therefore everything else being equal, the coarser grained soils should have higher C.B.R. values than the finer grained soils.

Then if one plots to some scale the values of C.B.R., obtained from tests run on the soaked sample, perpendicular to the best straight line on the classification chart,(C.B.R. for one depth one location opposite the point on the classification chart for soil of same location and depth), some relation should exist. This plot was made for Grande Prairie and Ft. St. John as shown in "Figures A and B" in the envelope in the back. As the classification tests were run on soils of depths 0-6", 6-12" and 12-18", and the C.B.R.

Tests only run on soils of depths 0-6 and 9-15 inches, the C.B.R. value for 0-6 inches was plotted opposite the classification test for 0-6 inches and the 9-15 inch C.B.R. value was plotted opposite both the 6-12 and 12-18 inch classification test point.

A study of these charts shows that no such relationship appears to exist. The C.B.R. values are higher as a rule for soils at the lower end of the line on the classification chart but the wide discrepancy in results renders this chart almost useless for the purpose of design. Therefore, no further attempt will be made to use this relationship.

4. Toughness

A measure of the toughness of the soil is given as Plasticity Index/Flow Index and its value is supposed to range between 1 and π, usually less than 3.

It is reasonable that there should be a relation between the toughness of the soil and the load bearing capacity. So a plot of Toughness vs $P_{0.5}$ (Load from the Subgrade Load Test at a deflection of 0.5 inches) has been plotted in Figures 19A and 19B for Grande Prairie. There is seen to be a wide discrepancy in the results and no relation is apparent. The inconsistency can probably be partly explained by the large variation in Flow Index values which often went to zero or were negative. This plot is therefore of no use for this project.

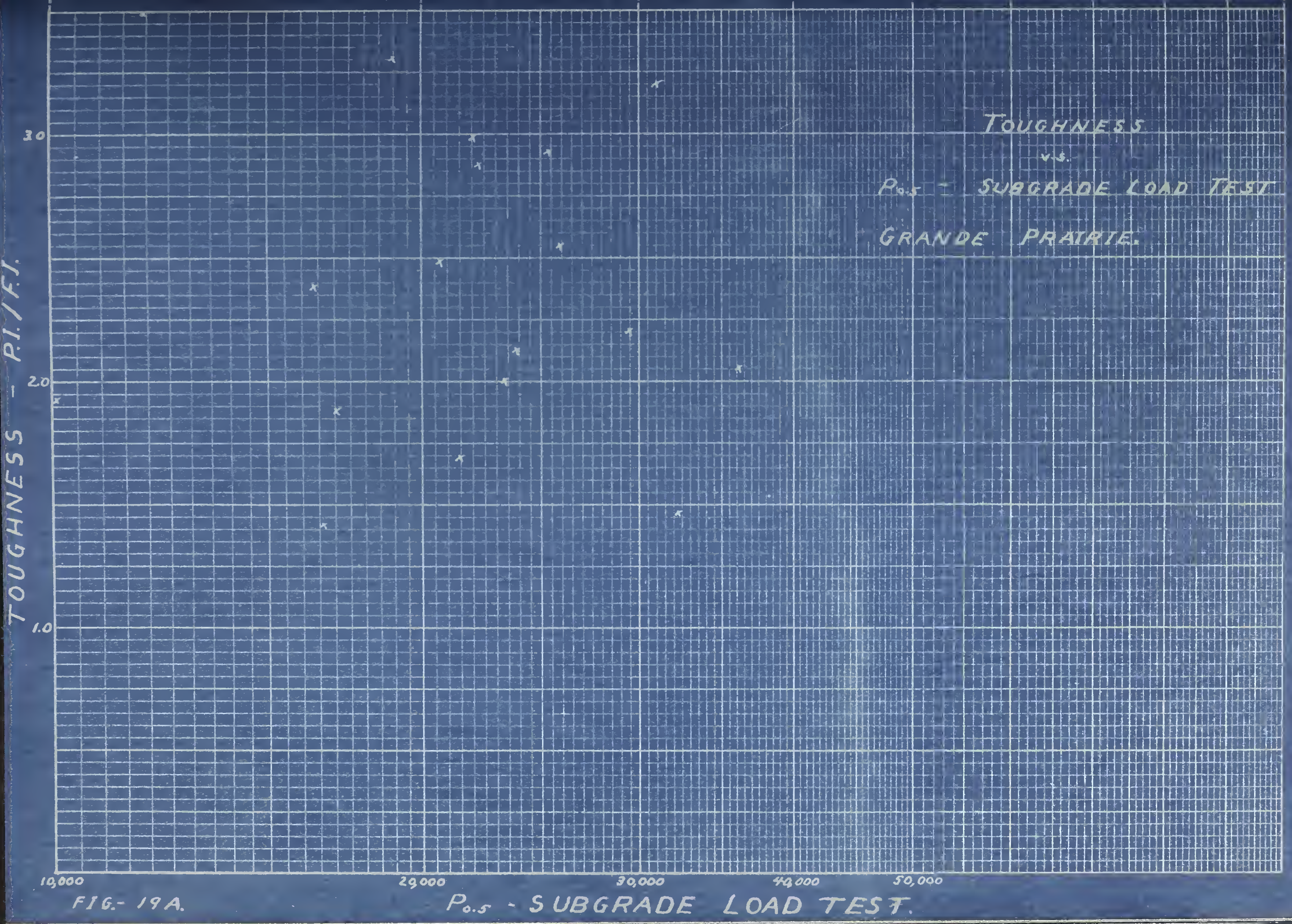

TOUGHNESS
v.s.
$P_{0.5}$ - SUBGRADE LOAD TEST
GRANDE PRAIRIE.
TOUGHNESS - P.I./F.I.
3.0
2.0
1.0
10,000
20,000
30,000
40,000
50,000
$P_{0.5}$ - SUBGRADE LOAD TEST.

FIG.- 19A.

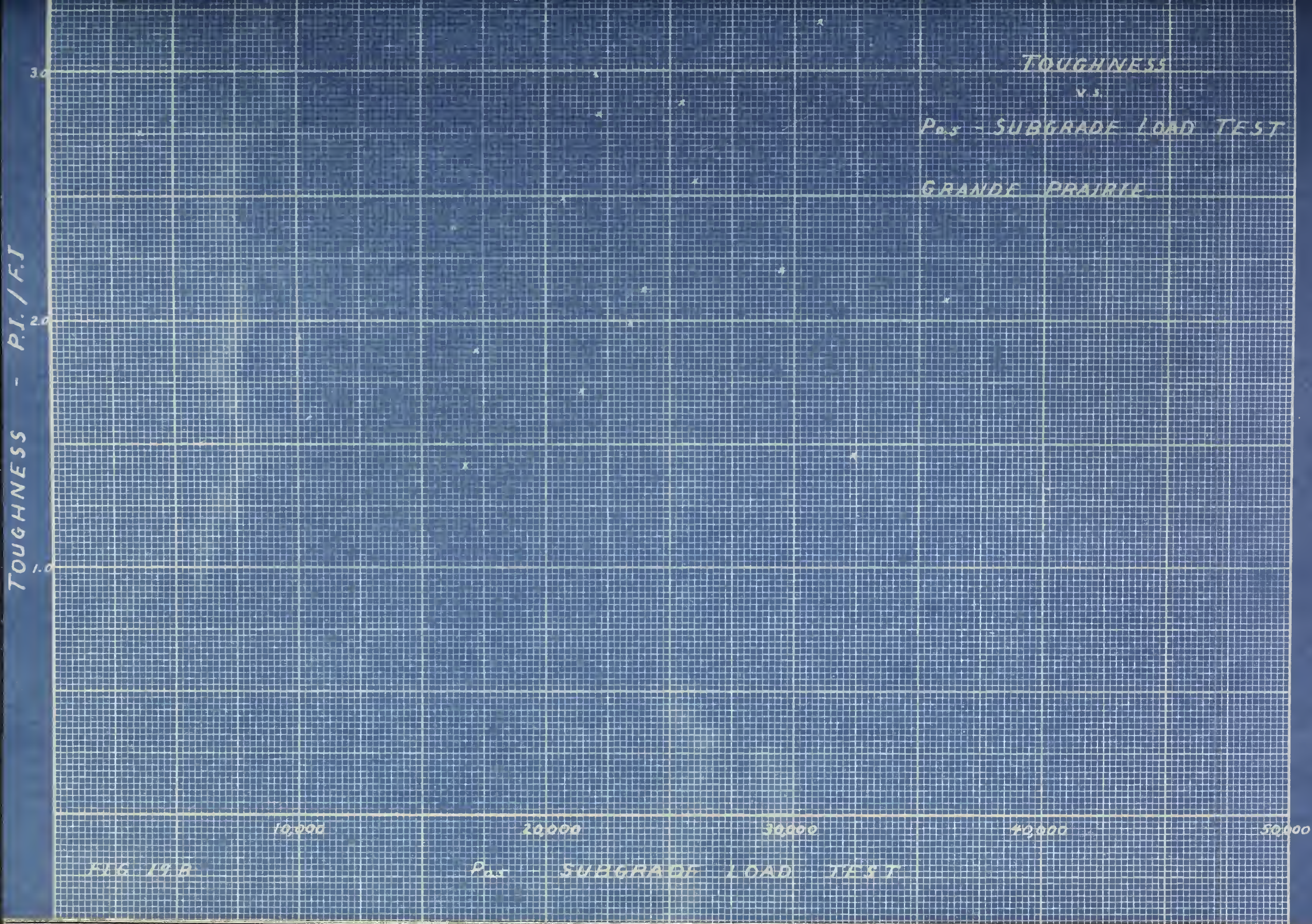

TOUGHNESS
v.s.
P0.5 - SUBGRADE LOAD TEST
GRANDE PRAIRIE
TOUGHNESS - P.I. / F.I.
3.0
2.0
1.0
10,000
20,000
30,000
40,000
50,000
P0.5 SUBGRADE LOAD TEST

FIG. 19 B

5. Moisture Content

The moisture content of a subgrade material probably has the greatest effect on its bearing capacity than any other single factor.

The natural moisture content of a subgrade can vary considerably and in a very short distance. The variation in a vertical direction can also be large.

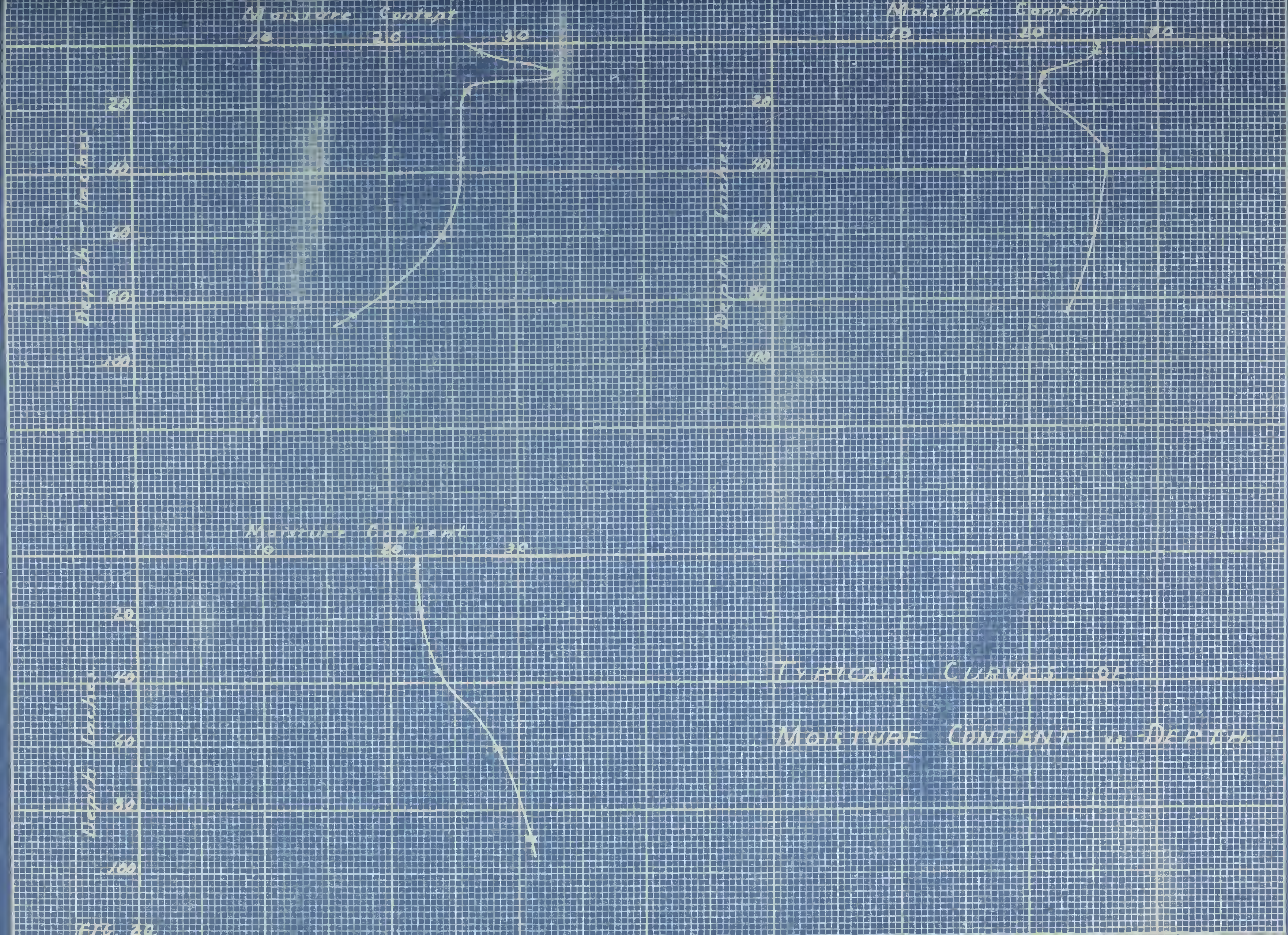
With the same soil type, over the range of moisture content in which subgrade materials generally exist, the soil has a higher bearing capacity the drier it becomes.

If all other variables other than moisture content are eliminated, there should be some relation (linear or otherwise) between natural moisture content and Modulus of Deformation as derived from the field test. A plot of this shall therefore be attempted on both arithmetic and semi logarithmic paper for soils of each type.

With the large variations in moisture content with depth the question arises as to what value for moisture content should be used in this plot. From a study of the stress distribution below a loaded plate, which has previously been computed, it can be seen that the stress has been distributed over a large enough area at 2 feet below the plate that the value of stress immediately below the plate at this depth is relatively small. Therefore the moisture content in the top 2 feet of subgrade material only will be considered.

Plots of maximum moisture content in top 2 feet, average moisture content in top 2 feet, and average moisture content in top 1 foot of the subgrade vs modulus of deformation for a deflection of 0.05 inches from the field test shall be made (Figures 21-34).

To obtain values of maximum moisture content in top 2 feet, average moisture content in top 2 feet and average moisture content in top 1 foot of subgrade, plots of moisture content vs depth were made for each undisturbed location and the required values picked off. Some typical curves of these are shown in Figure 20.



TYPICAL CURVES OF

MOISTURE CONTENT vs DEPTH

FIG. 20

Moisture Contents - Grande Prairie

Runway	Station	Offset	Max.Moist.Cont. in First 2'	Ave.Moist.Cont. in First 1'	Ave.Moist.Cont. in First 2'
#11	14+00	33' L	33	30	29
	17+22	CL	30	25	27
	18+00	33' L	29	26	27
		33' R	32	31	30
	20+50	33' L	25	22	23
		33' R	27	25	23
	23+00	33' L	20	18	18
		33' R	20	19	18
	29+00	33' L	26	25	23½
		33' R	22	21	18
	35+00	33' L	25	20	18
		33' R	20	18	17
	40+50	CL	25	23	21
	45+00	33' L	18	18	18
		33' R	27	24	23

Moisture Contents - Grande Prairie

Runway	Station	Offset	Max.Moist.Cont. in First 2'	Ave.Moist.Cont. in First 1'	Ave.Moist.Cont. in First 2'
T.S. #4	5+50	25' R CL	24	21	20
T.S. #3	8+80	35' L CL	24	22	21
		35' R CL	23	21	22
T.S. #2	8+10	50' L (EEPA)	26	19	22
	5+10	20' L	23	23	21
	1+60	68' L	30	23	21
	4+80	250' L	17	14	15
	1+60	250' L	18	16	16
T.S. #1	4+40	CL D.O.T.	24	19	18
	3+70	180' R (LS)	21	18	16
	2+70	60' R (LS)	38	33	32

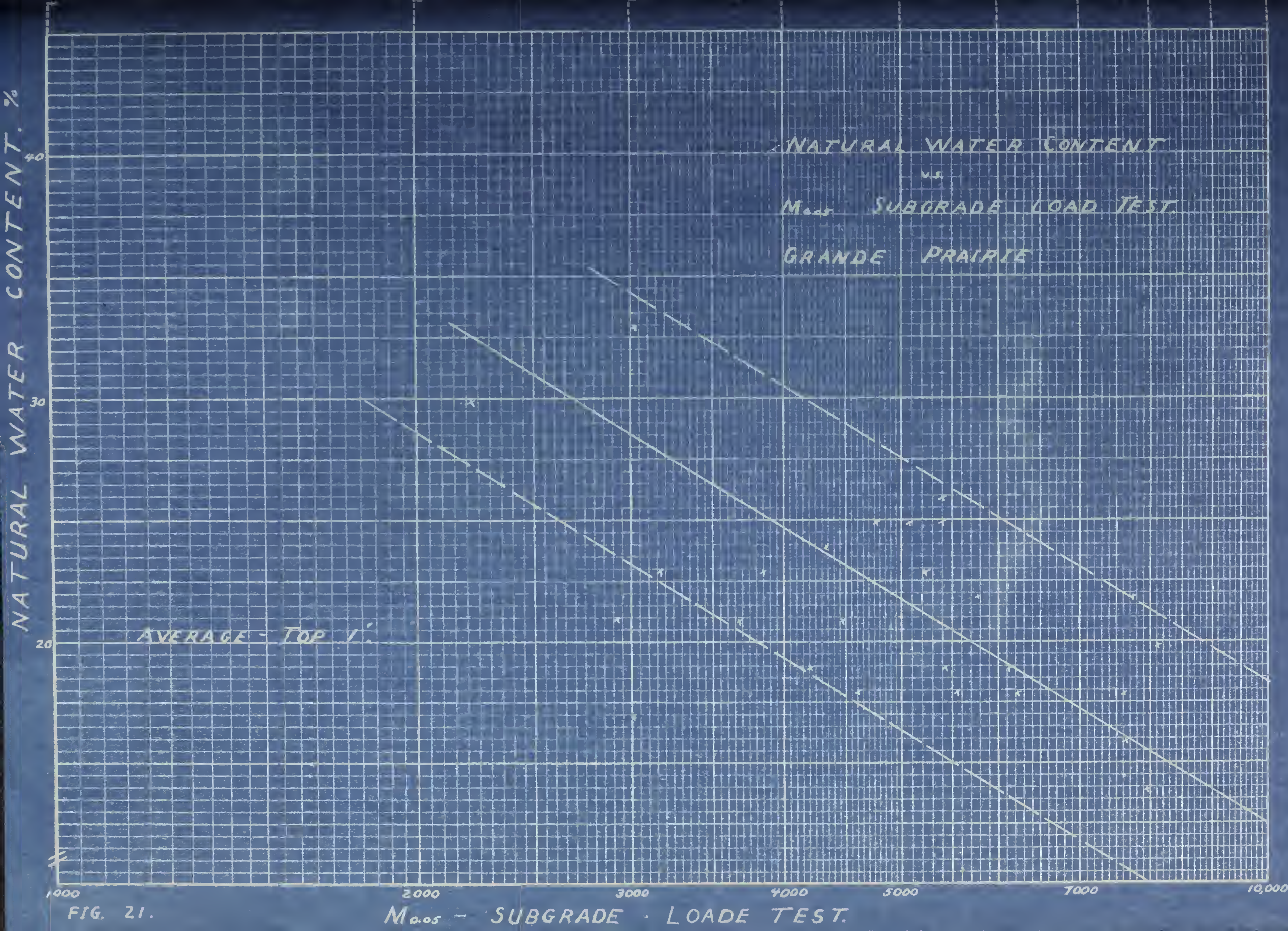

NATURAL WATER CONTENT
vs.
$M_{0.05}$ SUBGRADE LOAD TEST.
GRANDE PRAIRIE
AVERAGE - TOP 1'.
NATURAL WATER CONTENT. %
40
30
20
1000
2000
3000
4000
5000
7000
10,000
$M_{0.05}$ - SUBGRADE · LOADE TEST.

FIG. 21.

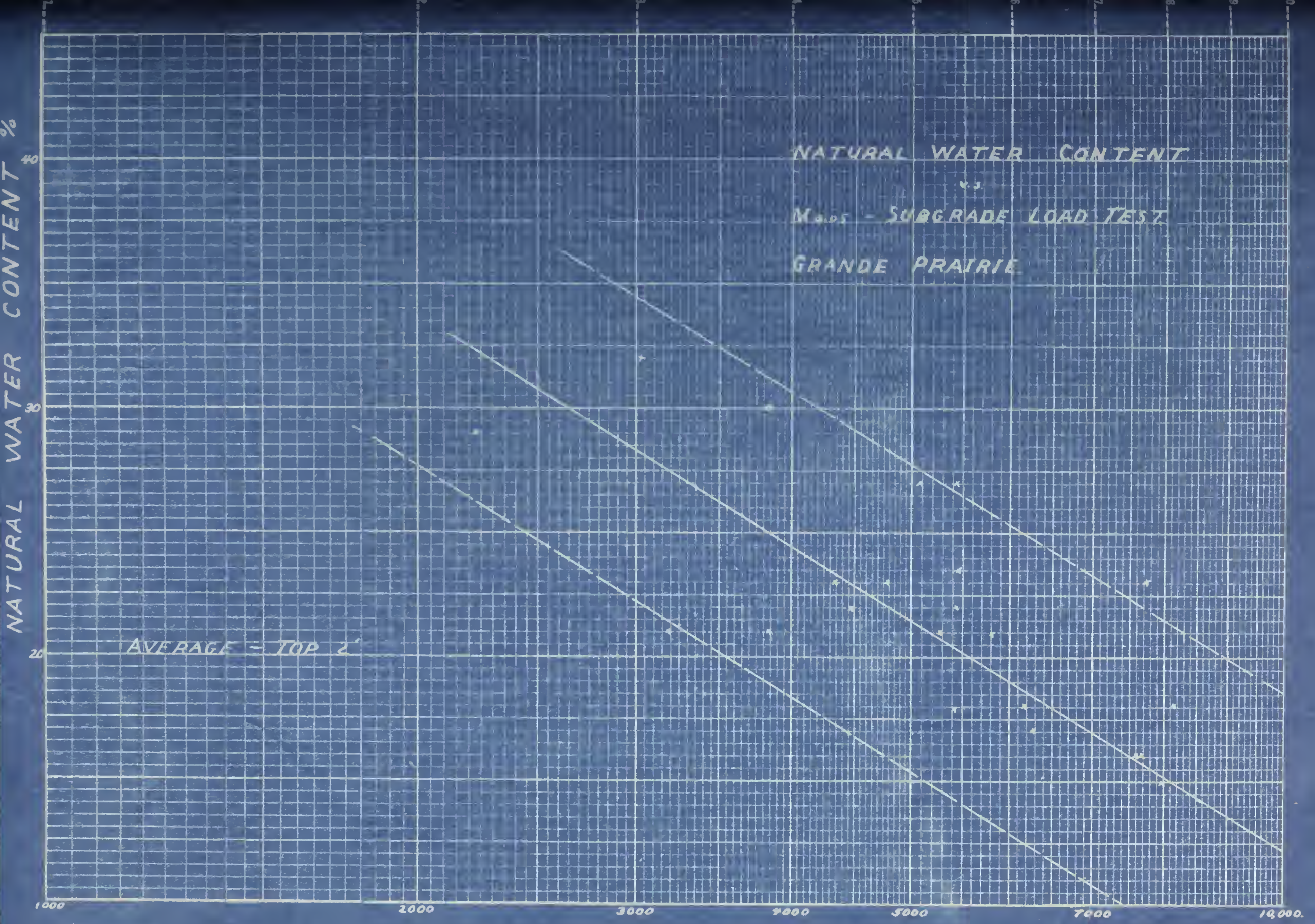

NATURAL WATER CONTENT
vs.
$M_{0.05}$ - SUBGRADE LOAD TEST
GRANDE PRAIRIE
AVERAGE - TOP 2'
NATURAL WATER CONTENT %
40
30
20
1000
2000
3000
4000
5000
7000
10,000.
$M_{0.05}$ - SUBGRADE LOAD TEST.

FIG. 22.

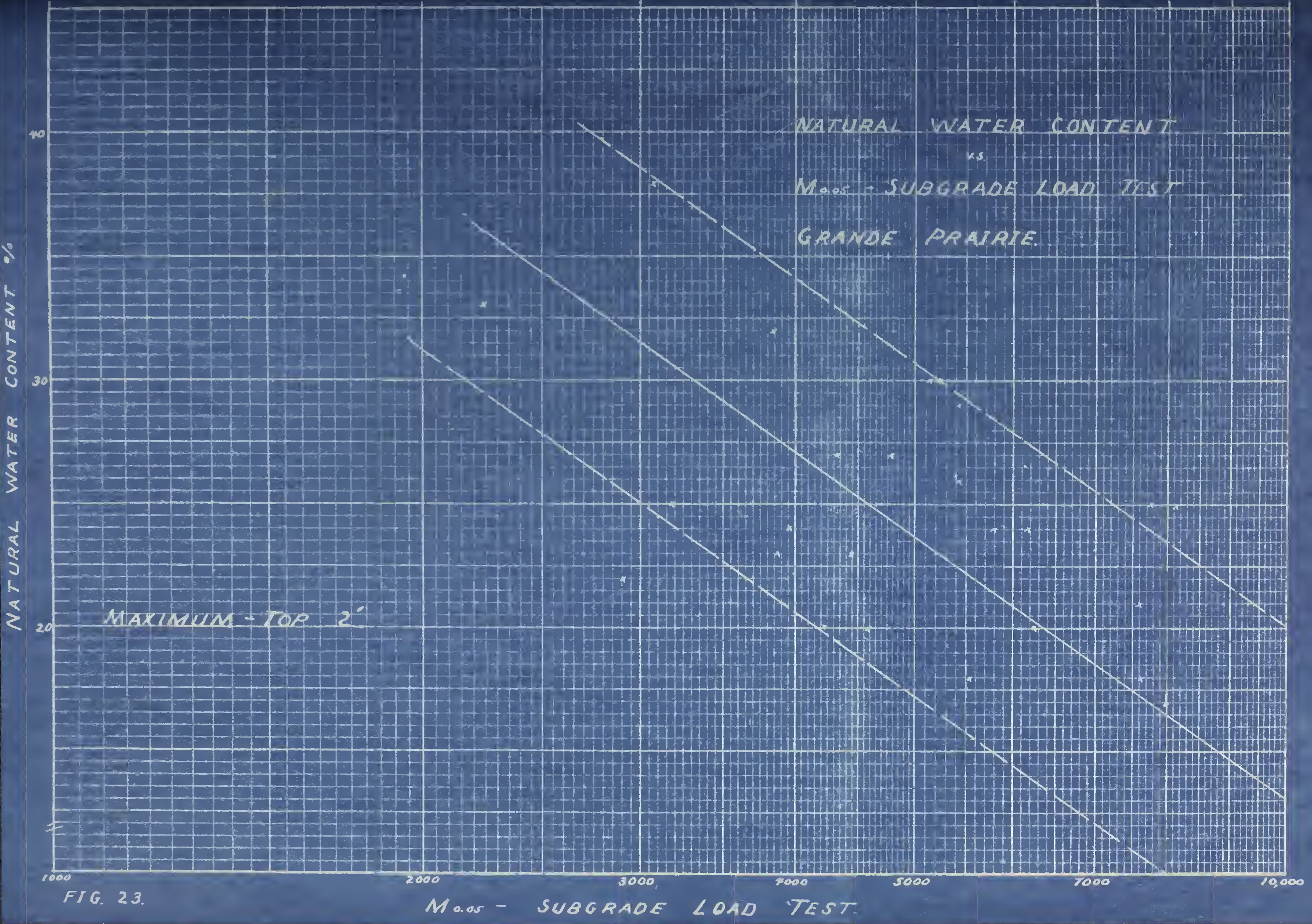

NATURAL WATER CONTENT
vs.
M0.05 - SUBGRADE LOAD TEST
GRANDE PRAIRIE.
MAXIMUM - TOP 2'
40
30
20
1000
2000
3000
4000
5000
7000
10,000
NATURAL WATER CONTENT %
M0.05 - SUBGRADE LOAD TEST

FIG. 23.

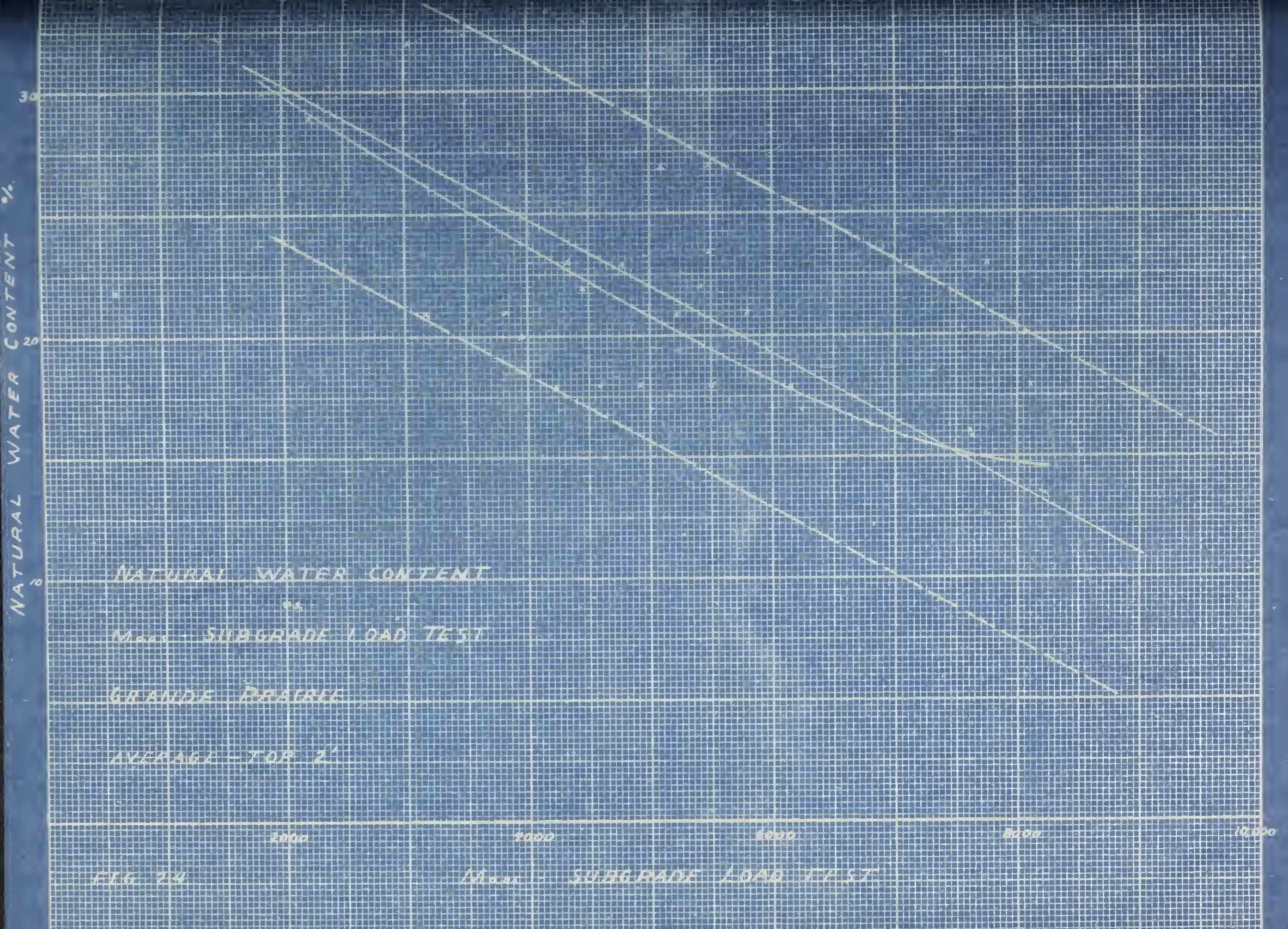

NATURAL WATER CONTENT
vs.
Max. SUBGRADE LOAD TEST
GRANDE PRAIRIE
AVERAGE TOP 2"
NATURAL WATER CONTENT %.
30
20
10
2000
4000
6000
8000
10,000
Max. SUBGRADE LOAD TEST

FIG. 24

Moisture Contents - Saskatoon Main

Runway	Station	Offset	Max.Moist.Cont. in First 2'	Ave.Moist.Cont. in First 1'	Ave.Moist.Cont. in First 2'
16 L	21+70	CL	25	21	22
	11+10	CL	23	20	21
	7+00	CL	21	13	18
	14+40	CL	23	21	22
	17+00	CL	27	24	25
	5+45	CL	20	18	17
7 L	13+75	CL	23	21	19
	18+25	CL	23	21	22
29 L	3+10	CL	30	27	27
	17+90	CL	23	22	21
	10+10	CL	24	23	23
7 R	19+10	CL	33	31	32
Conn. Strip	2+60	CL	25	25	25
16 R	13+80	CL	30	24	26
	15+05	27' R	33	27	29
	17+25	CL	26	23	24
	20+53	CL	23	14	18
29 R	26+10	CL	28	24	25
16 R	9+84	CL	37	29	32

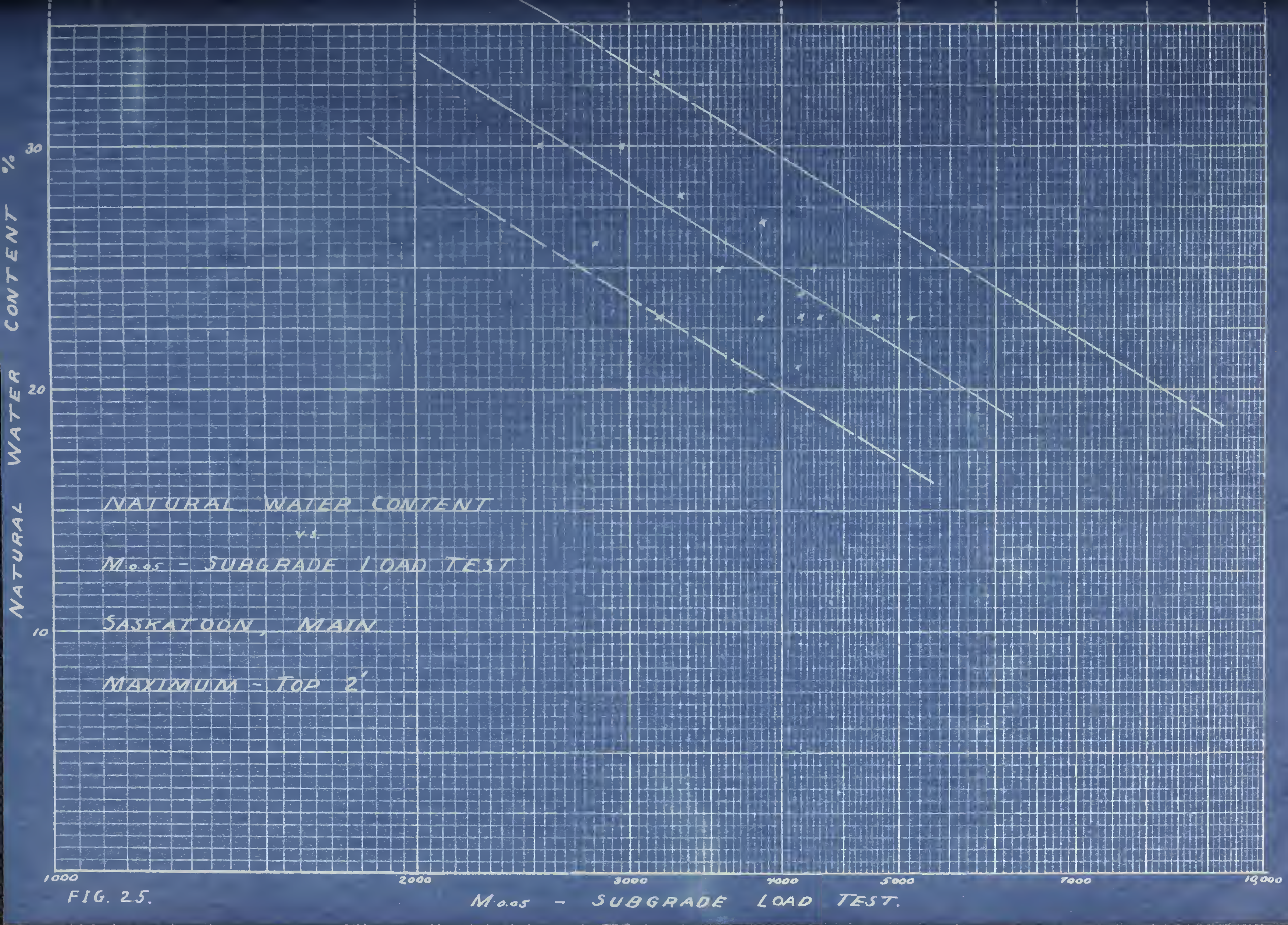

NATURAL WATER CONTENT
VS.
$M_{0.05}$ - SUBGRADE LOAD TEST
SASKATOON, MAIN
MAXIMUM - TOP 2'
NATURAL WATER CONTENT %
30
20
10
1000
2000
3000
4000
5000
7000
10,000
$M_{0.05}$ - SUBGRADE LOAD TEST.

FIG. 2.5.

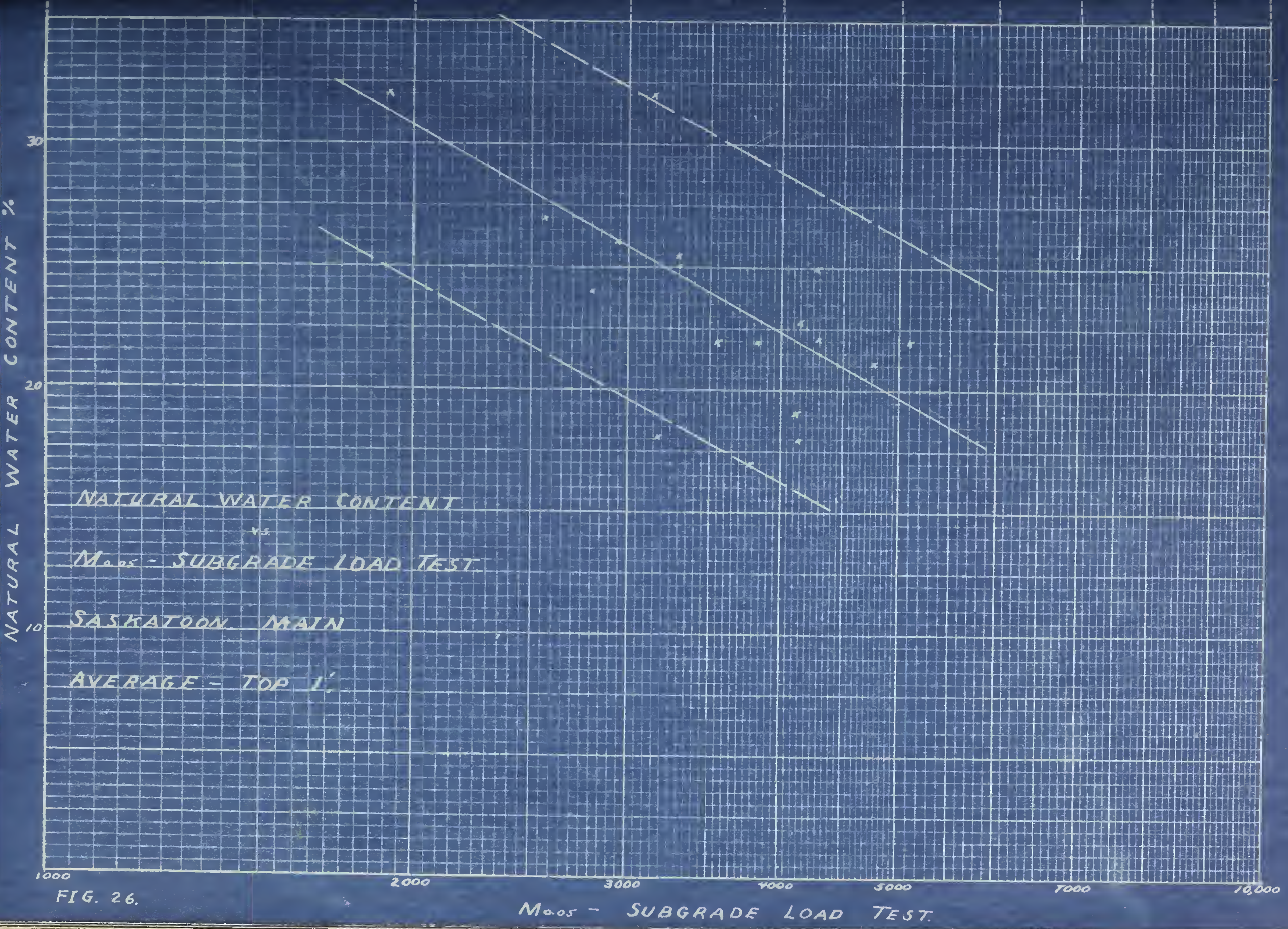

NATURAL WATER CONTENT
vs.
M0.05 - SUBGRADE LOAD TEST
SASKATOON MAIN
AVERAGE - TOP 1'
NATURAL WATER CONTENT %
30
20
10
1000
2000
3000
4000
5000
7000
10,000
M0.05 - SUBGRADE LOAD TEST.

FIG. 26.

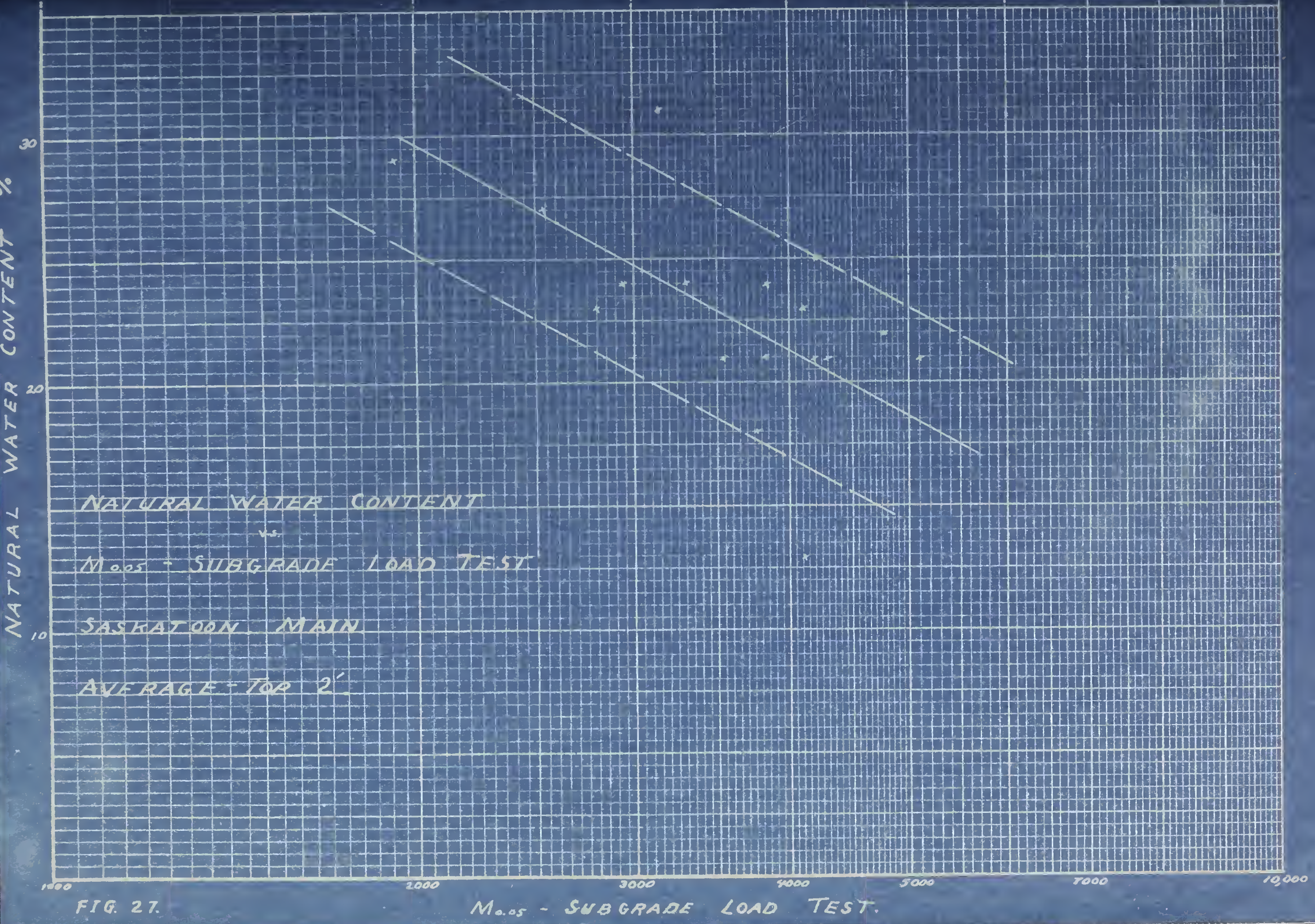

NATURAL WATER CONTENT vs. $M_{0.05}$ - SUBGRADE LOAD TEST
SASKATOON MAIN
AVERAGE - TOP 2'
NATURAL WATER CONTENT %
30
20
10
1000
2000
3000
4000
5000
7000
10,000
$M_{0.05}$ - SUBGRADE LOAD TEST.

FIG. 27.

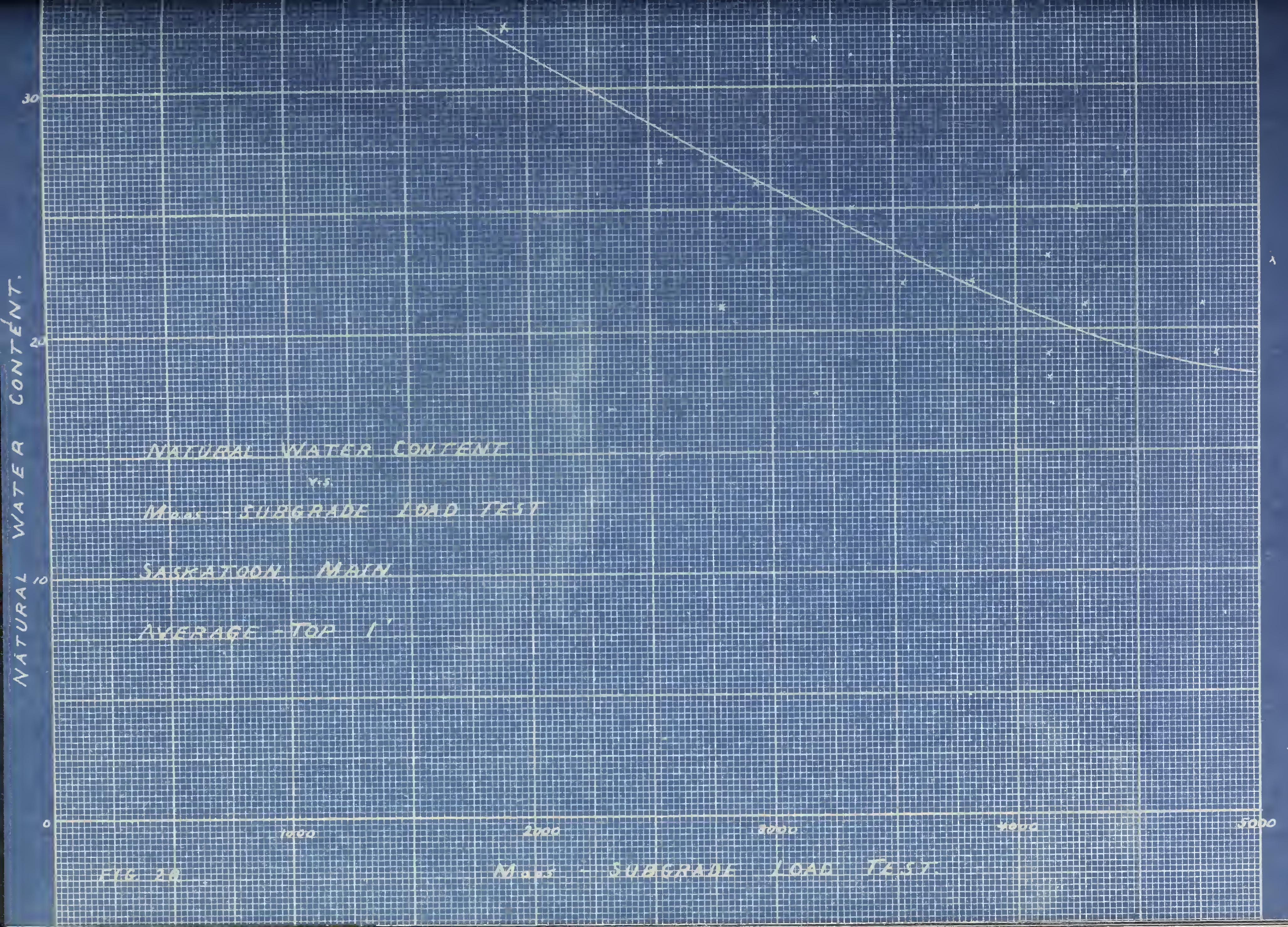

NATURAL WATER CONTENT
vs.
Mass SUBGRADE LOAD TEST
SASKATOON MAIN
AVERAGE - TOP 1"
NATURAL WATER CONTENT.
30
20
10
0
1000
2000
3000
4000
5000
Mass - SUBGRADE LOAD TEST.

FIG. 28

Moisture Contents - Stevenson's Field, Winnipeg

Runway	Station	Offset	Max.Moist.Cont. in First 2'	Ave.Moist.Cont. in First 1'	Ave.Moist.Cont. in First 2'
N.E. #35	3+27	CL	30	28	29
	6+70	CL	35	30	32
	11+08	CL	34	33	34
	15+00	CL	27	27	27
	19+07	CL	26	25	25
	23+14	CL	36	28	31
E.W. #26	13+20	CL	36	32	33
	21+20	CL	37	34	35

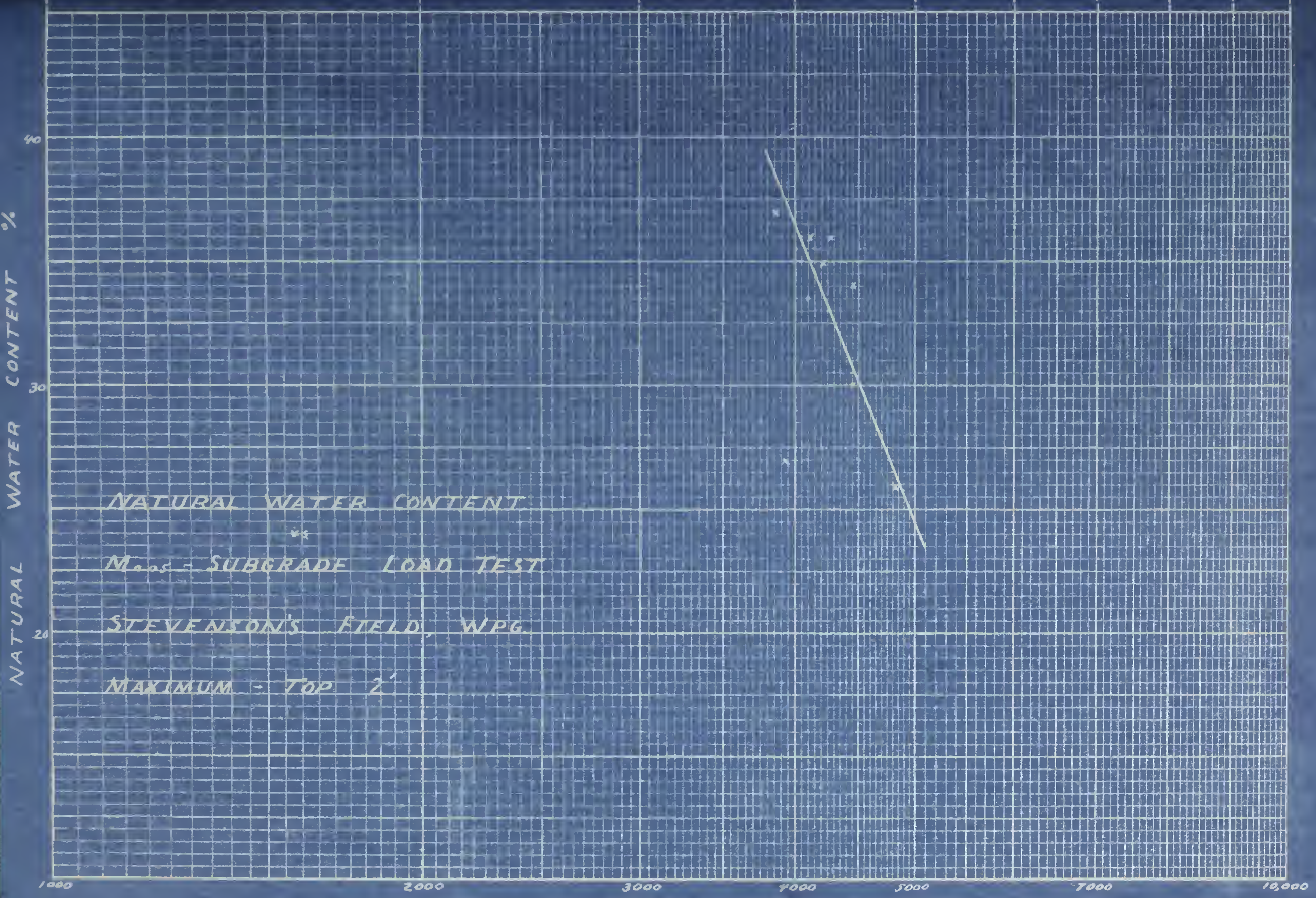

NATURAL WATER CONTENT
vs
$M_{.05}$ - SUBGRADE LOAD TEST
STEVENSON'S FIELD, WPG.
MAXIMUM - TOP 2'
NATURAL WATER CONTENT %
40
30
20
1000
2000
3000
4000
5000
7000
10,000
$M_{.05}$ - SUBGRADE LOAD TEST.

FIG. 29.

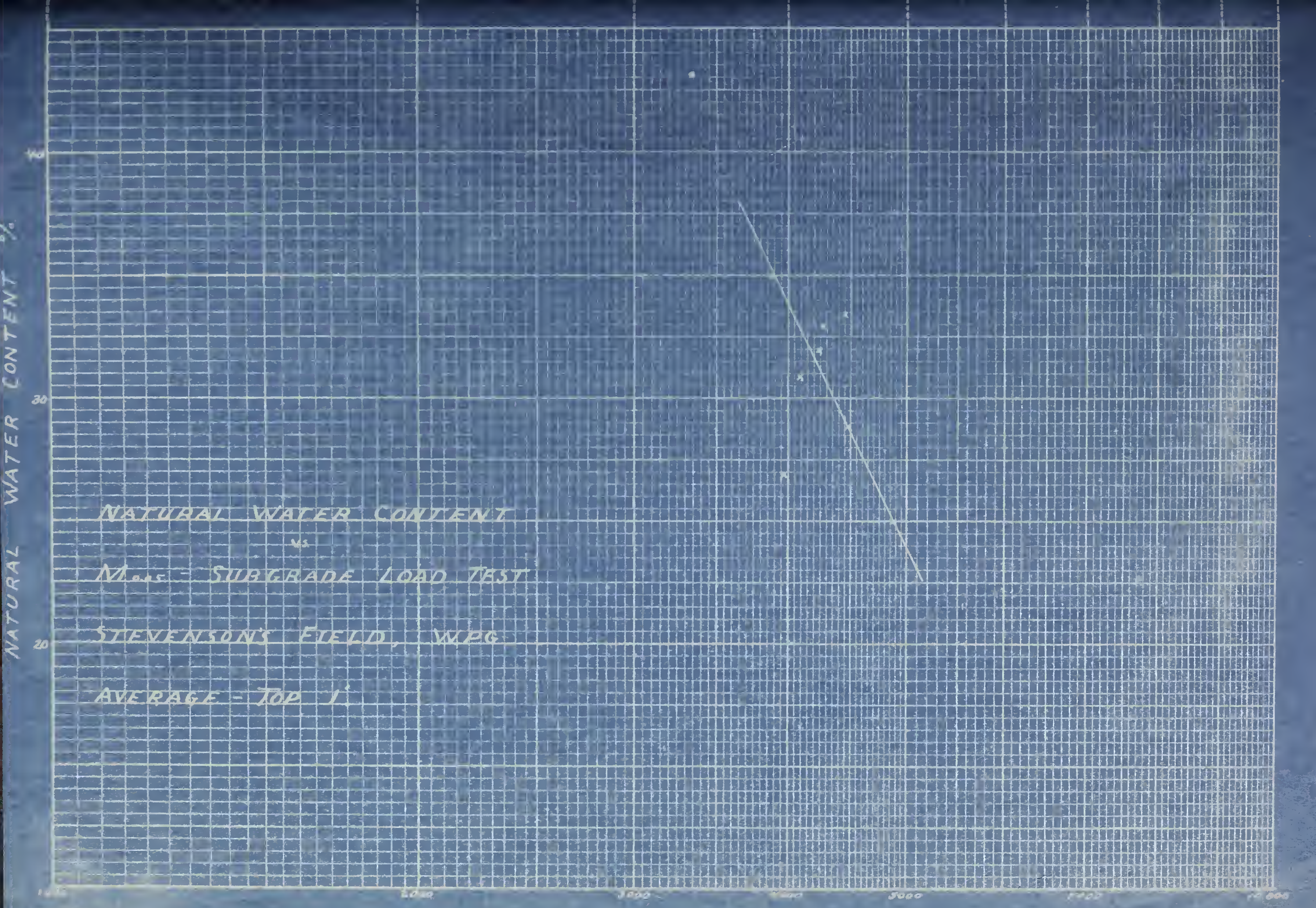

NATURAL WATER CONTENT
vs
M.25 - SUBGRADE LOAD TEST
STEVENSON'S FIELD, WPG
AVERAGE - TOP 1'
NATURAL WATER CONTENT %
40
30
20
1000
2000
3000
4000
5000
7000
10 000
M.25 - SUBGRADE LOAD TEST.

FIG. 30.

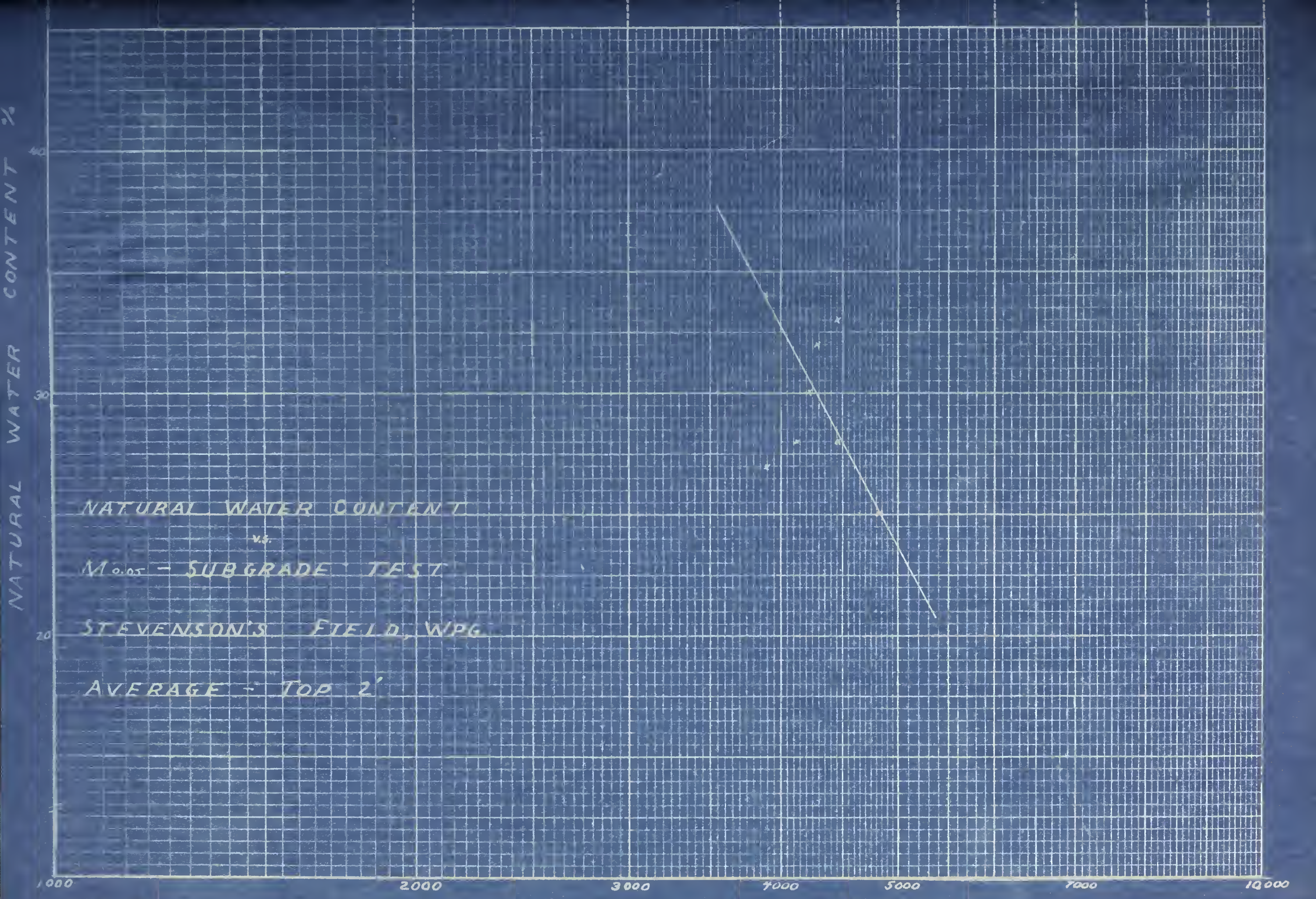

NATURAL WATER CONTENT
v.s.
$M_{0.05}$ - SUBGRADE TEST
STEVENSON'S FIELD, WPG.
AVERAGE - TOP 2'
NATURAL WATER CONTENT %
40
30
20
1000
2000
3000
4000
5000
7000
10000
$M_{0.05}$ - SUBGRADE LOAD TEST.

FIG. 31.

Moisture Contents - Lethbridge

			Natural Moisture Content		
Runway	Station	Offset	Ave.Top 1'	Ave.Top 2'	Max.Top 2'
NW-SE	4+00	CL	13	13	14
	13+00	CL	16	14	16
	20+00	CL	13	14	17
	29+00	CL	15	13	19
NE-SW	7+00	CL	13	12	14
	12+50	CL	14	15	16
	22+00	CL	10	10	12
	31+00	CL	11	11	13
NE-SW Taxi	5+00	CL	18	18	21
Con. Strip NE-SW & EW	7+00	CL	17	15	20
	9+50	CL	15	15	16
	4+00	CL	10	9	11
Apron	2+85	75' E	11	10	14
	5+47	112' E	14	13	14
	8+00	37.5' E	10	10	10
	12+25	40' E	9	8	10
	7+00	37.5' E	11	8	13
	9+00	115' E	15	15	16

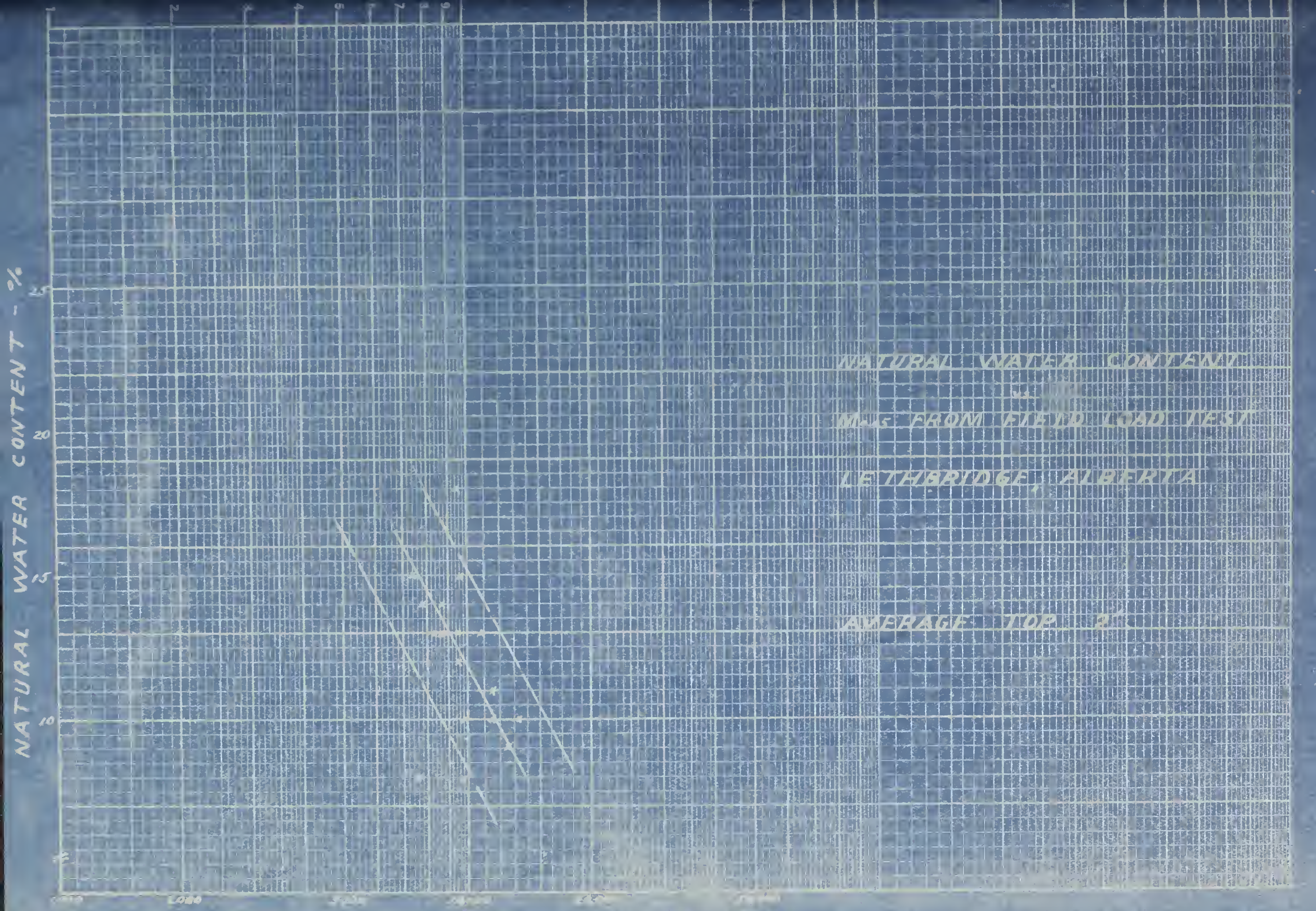

NATURAL WATER CONTENT
vs
Mₑₛ FROM FIELD LOAD TEST
LETHBRIDGE, ALBERTA
AVERAGE TOP 2'
NATURAL WATER CONTENT - %
25
20
15
10
5
Mₑₛ FROM FIELD LOAD TEST

FIG. 32.

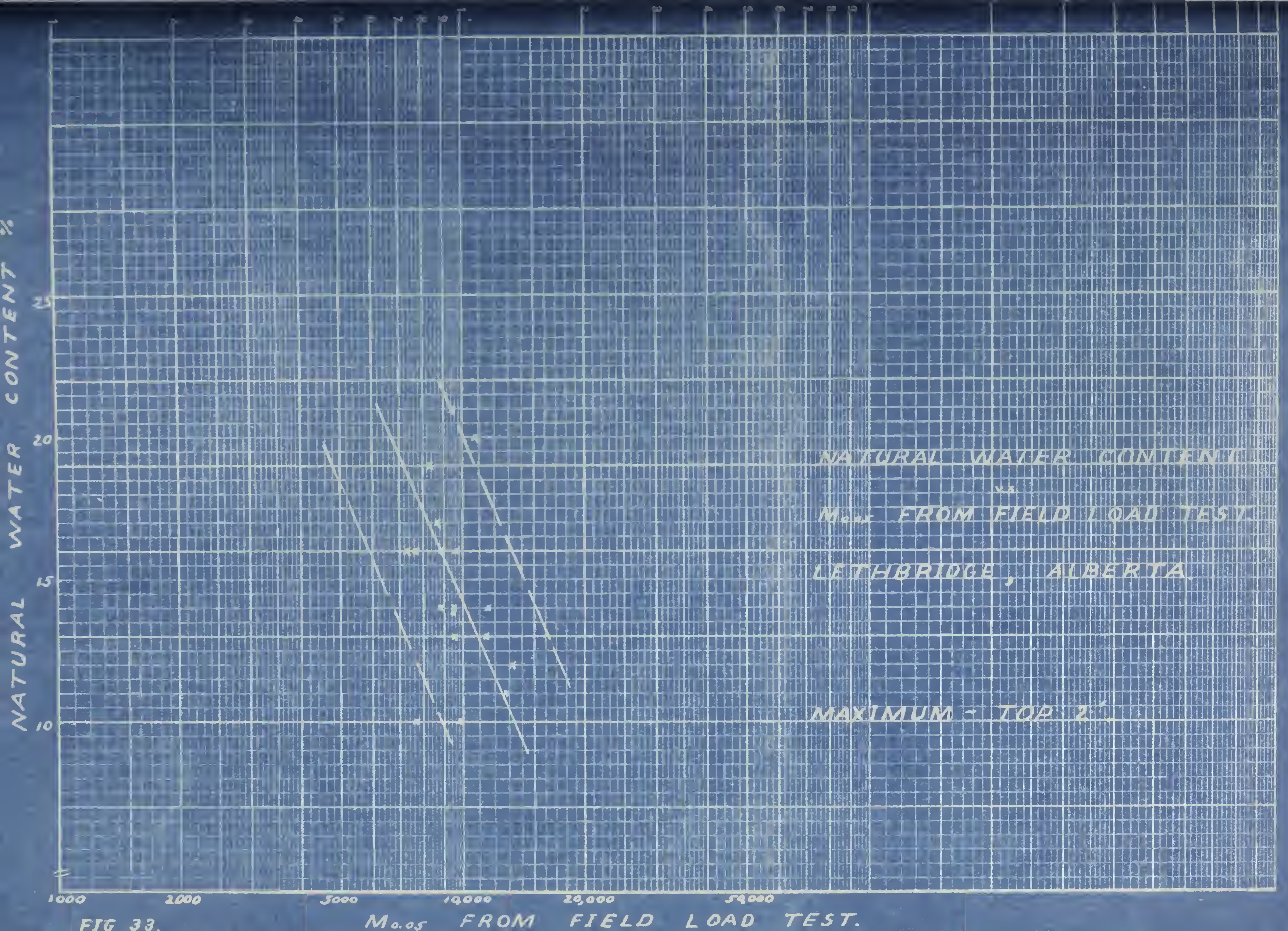

NATURAL WATER CONTENT
vs
$M_{0.05}$ FROM FIELD LOAD TEST
LETHBRIDGE, ALBERTA
MAXIMUM - TOP 2'
NATURAL WATER CONTENT %
25
20
15
10
1000
2000
5000
10,000
20,000
50,000
$M_{0.05}$ FROM FIELD LOAD TEST.

FIG 33.

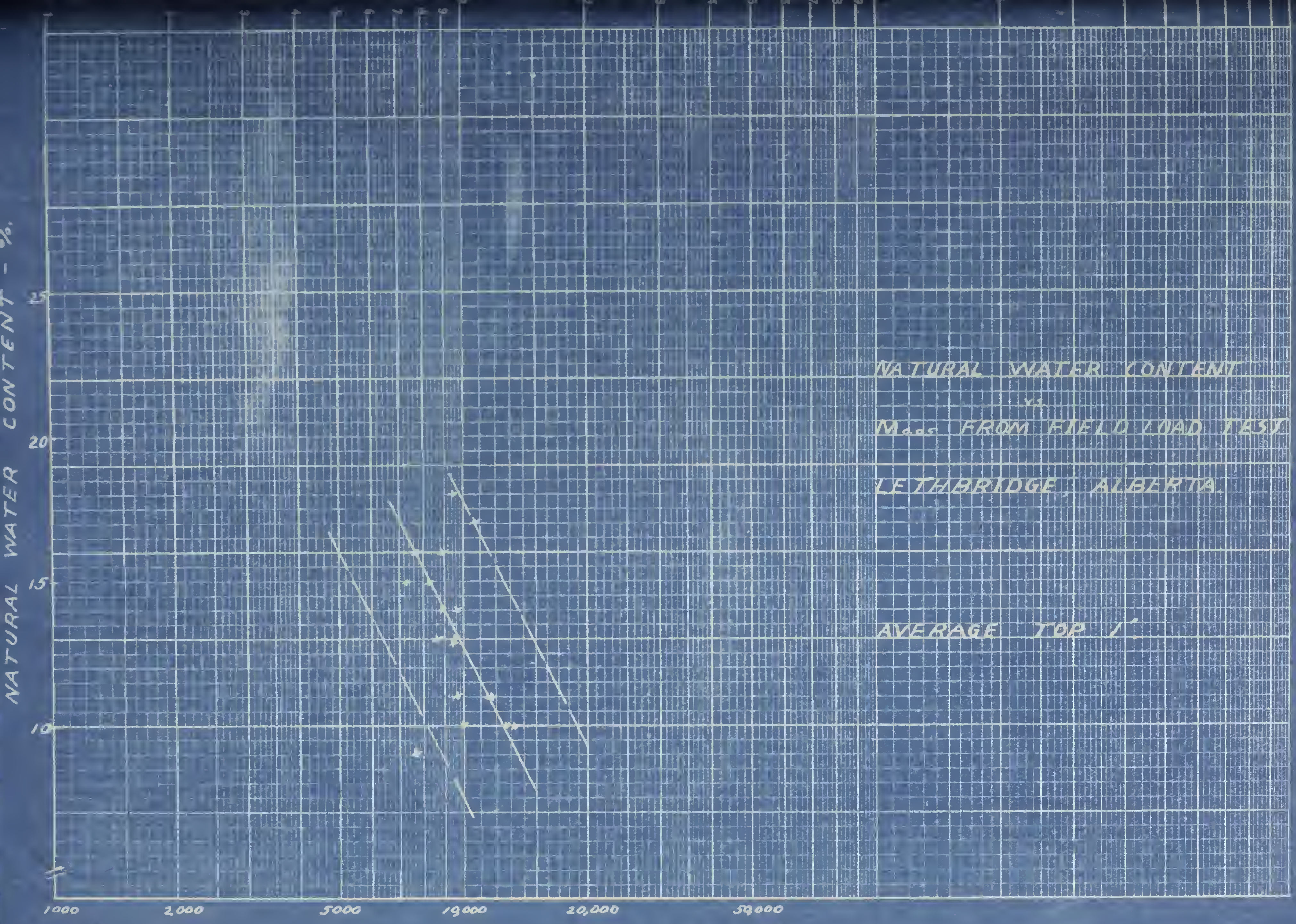

NATURAL WATER CONTENT
vs
$M_{0.05}$ FROM FIELD LOAD TEST
LETHBRIDGE, ALBERTA
AVERAGE TOP 1'
NATURAL WATER CONTENT - %.
25
20
15
10
1000
2000
5000
10,000
20,000
50,000
$M_{0.05}$ FROM FIELD LOAD TEST.

FIG. 34.

The plot of natural water content vs Log "$M_{0.05}$" from subgrade load test, seems to give the best straight line relationship. This indicates that the strength of the subgrade increases rapidly with decrease in water content, emphasizing the importance of good control on the water content ie. good drainage.

A relationship between these 2 quantities is seen to definitely exist, but the points do not all lie on a straight line. Instead they appear to lie within a zone.

The graph of maximum moisture content in top 2 feet vs. $M_{0.05}$ appears to give the best results, that is the points fall within a narrower zone. This indicates that most of the deflection encountered in the load test on the subgrade is taken up by the section within the top 2 feet which has the highest moisture content.

These curves again show the importance of having a different design chart, formula or whatever is to be used for each type of soil. The zoning of the classification chart as suggested earlier is probably as good a method of dividing soils into different classes as any.

Curves of this sort could not be used to accurately determine the strength of a subgrade but they could be used to indicate its strength and as a check against other work. Therefore it appears that they are worth while and should be used to check the other work when evaluating a subgrade.

The importance of good drainage of the runways is readily shown by these plots. If the moisture content of the subgrade can be maintained throughout the entire year at a low value, the subgrade strength is higher and consequently the base course thickness required is less.

Design of Airport Runways (i.e. Base Course plus Surface Thickness)

From the preceding work it appears that the best way to evaluate a subgrade material as to strength is as follows:

1. Obtain a reasonable number of samples of the subgrade material to be used and run classification tests on them. Thus the zone into which this material belongs can be determined.
2. A reasonable number of undisturbed samples of the subgrade material should be obtained and triaxial compression tests run on them. Samples from at least 2 different depths from each location should be taken. Average values of Modulus of Deformation for a lateral pressure of 15 p.s.i. should be computed. Then by going to the chart for soil from the proper zone the value of "$M_{0.05}$", which is the subgrade strength as figured from a field test, can be picked off.

The problem now remains to determine the thickness of base course plus pavement that would be required to spread a predetermined surface load enough so the subgrade would be strong enough to hold up.

A plot similar to the design curves based on the California Bearing Ratio, substituting values of $M_{0.05}$ for the values of C.B.R., should give a set of design curves if there is a relationship between subgrade strength, surface load and thickness of pavement plus base course. It appears essential that some such relationship should exist.

As at the time that this paper was written only a limited number of triaxial tests had been run, and as the charts of $M_{0.05}$ from field test vs "M" from triaxial gave rather good results, the values of $M_{0.05}$ from the field tests will be used here. At every location a load test on the surface of the pavement was run, so the values for wheel load will be those obtained from surface load tests for ½ inch deflection.*
The value of "z" will be equal to the combined depth of pavement plus base course.

Curves of $M_{0.05}$ vs depth for varying wheel loads will be drawn up (Figure 35).

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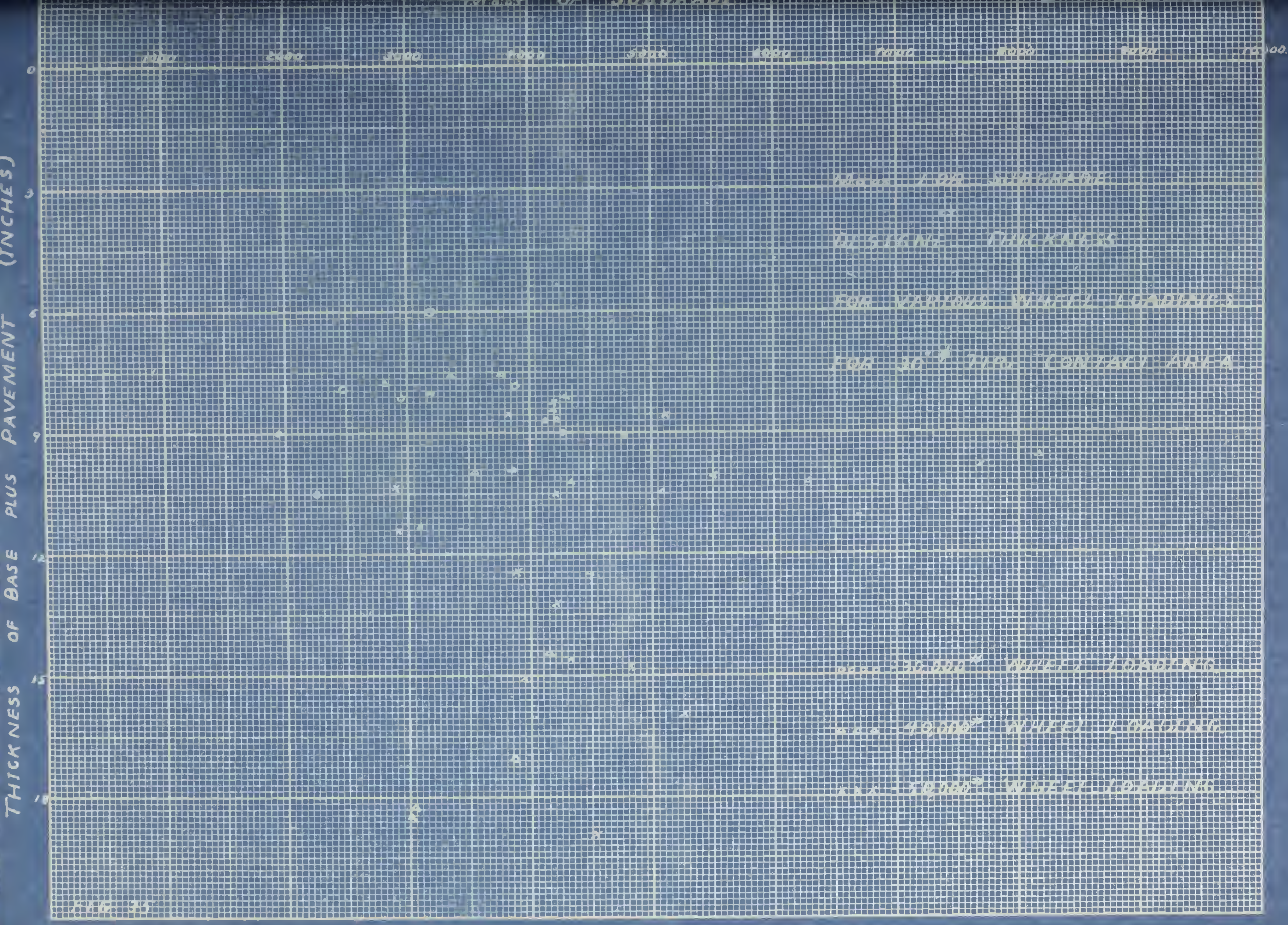
* This value of ½" deflection as being the limiting load is as was suggested by Dr. N.W. McLeod of Imperial Oil Ltd.


1000
2000
3000
4000
5000
6000
7000
8000
9000
10000.
0
3
6
9
12
15
18
THICKNESS OF BASE PLUS PAVEMENT (INCHES)
[illegible] SUBGRADE
DESIGN THICKNESS
FOR VARIOUS WHEEL LOADINGS
FOR 30 # TIRE CONTACT AREA
30,000# WHEEL LOADING
40,000# WHEEL LOADING
50,000# WHEEL LOADING
FIG. 35

There is such a scattering of points and lack of existence of any relationship on this graph that there must be something wrong with one or more sets of results that were used in the plot.

The author is of the opinion that the error is in the values for load bearing capacity of the runway as derived from the surface load test. In the running of this test in the field it was assumed that the temperature of the pavement had no effect on the deflection of the pavement.

This assumption appears to the author to be highly erroneous. He observed in the field at the time the tests were run that on a hot day the plate would shear through the pavement while on a cold day there would be little or no indication of shear failure, but a general settlement of the surrounding area. It was also observed that surface tests run near each other, one in the morning and the other in the afternoon, gave vastly different bearing capacities for the pavement in some cases.

At some locations, tests on the surface of the base produced higher bearing capacities than did surface tests at the same location on hot days.

As the temperature varied considerably from month to month, from day to day and even from morning to afternoon of the same day, it appears that the bearing capacities as derived from the surface load tests are not of too much value at this point. They will therefore be disregarded.

The author is now of the opinion that the test that should have been run at each location was the test called "The Base Course Load Test". This test was run on the surface of the base course in the centre of a 12 ft. diameter area from which the pavement had been carefully stripped. Variations in temperature obviously would not have much effect on the bearing capacity of a compacted gravel base. Again here the load required to produce $\frac{1}{2}$ inch deflection of the plate shall be considered as the bearing capacity of the runway as run on the surface of the base.

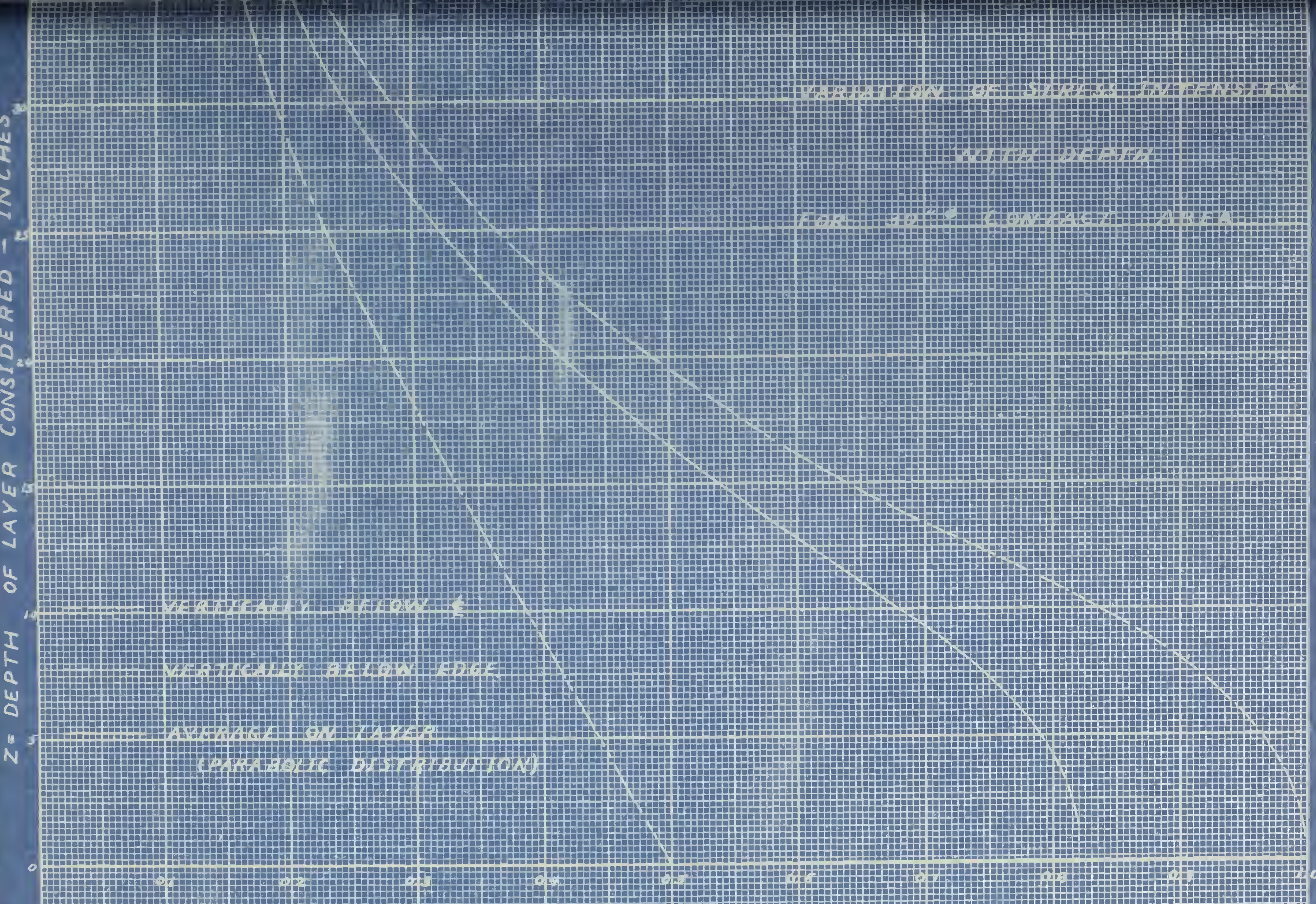


FIG. 36 VALUES OF "C" (FROM EQUATION $\sigma = CP$)

From the meagre data from Base Course Load Tests that is available an indication at least as to the value of "K" in the preceding equation "$CP = KM_{0.05}$", can be determined.

$M_{0.05}$ is known from the Subgrade Load Tests

P is known from the Base Course Load Tests

C is known from the preceding graph

Thus the values of "K" for the different airports can be computed and tabulated for each location at which a Base Course Load Test was run.

Stevenson's Field, Winnipeg, or soil in Zone "D"

Location	P	Z	$M_{0.05} \times K$	$M_{0.05}$	K
#35 6+86 CL	30,200	9	.710P = 21400	4210	5.10
#35 15+08 CL	30,800	9½	.695P = 21400	3950	5.42
#35 23+04 CL	34,000	10	.682P = 23200	4140	5.58
				Average	5.37

Grande Prairie, or soil in Zone "C"

Location	P	Z	$M_{0.05} \times K$	$M_{0.05}$	K
#11 17+20 CL	34,100	7½	.748P = 25500	5080	5.02
#11 20+50 33'L	48,300	8	.738P = 35600	7760	4.58
#11 29+00 33'L	50,000	6½	.768P = 38400	5440	7.06
#11 35+00 33'R	52,500	7	.758P = 39800	6260	6.36
T.S.4 5+50 25'R	35,000	8	.738P = 25800	3960	6.52
T.S.2 5+10 20'R	34,000	11	.652P = 22300	3840	5.78
T.S.2 1+60 250'L	88,000	16	.518P = 45600	7660	5.96
T.S.1 3+70 180'L	88,100	16½	.502P = 44200	7620	5.79
				Average	5.88

Lethbridge, or soil from Zone A

Location			P	Z	$M_{0.05} \times K$	$M_{0.05}$	K
NW-SE	4+00	CL	93,700	5	.792P = 74200	9540	7.78
NE-SW	12+50	CL	67,000	7	.758P = 50800	9630	5.26
Apron	12+50	40' E	78,000	5	.792P = 61800	7650	8.07
						Average	7.04

Saskatoon Main, or soil from Zone B

Location		P	Z	$M_{0.05} \times K$	$M_{0.05}$	K
16L	21+60	47,500	6	.776P = 36800	3840	9.60
16L	6+90	32,800	5½	.785P = 25800	4140	6.22
16R	10+00	19,500	6	.776P = 15100	1920	7.87
29L	10+00	35,500	6½	.768P = 27300	4140	6.60
29L	3+00	34,300	5	.792P = 27200	2540	7.88
EECS	2+50	40,200	6½	.768P = 30800	4240	7.30
					Average	7.53

From this data it is obvious that there is a different value of "K" for each type of soil, so design charts will now be plotted for subgrade materials lying in each zone.

As there are only 3 values of "K" for soil in zone A and they are so widespread and in the range of those for soil in Zone B, the average value of "K" for Zone B will be assumed to be the average value for both, Zone A and B. Thus only one set of design curves will be plotted for Subgrades in "Zone A and B".

Another set will be plotted for subgrades in "Zone C".

A final set will be plotted for subgrades in "Zone D".

These design curves will be plotted after the same style as the California Bearing Ratio curves have been drawn up with the substitution of "$M_{0.05}$ for the Subgrade" for "Values of C.B.R.". That is values of "$M_{0.05}$" for the subgrade will be plotted against thickness of base required for each of several wheel loads.

The following design curves have only been drawn up for a 30 inch diameter tire contact area and similar curves would have to be drawn for different tire contact areas.

These curves are only tentative design curves as only a very limited amount of data was available for each airport. Therefore they are perhaps rather theoretical and should be checked by further testing and changed if the change is found necessary.

Subgrades from Zone A and B $\qquad M_{0.05} = \frac{P \times C}{7.53}$

P	$M_{0.05}$ for z =							
	3	6	10	15	20	25	30	35
5,000	550	517	403	360	279	216	163	135
10,000	1090	1030	907	718	558	433	325	269
20,000	2170	2070	1810	1440	1120	865	643	538
30,000	3260	3100	2720	2130	1670	1290	975	803
40,000	4340	4130	3630	2940	2230	1730	1300	1070
50,000	5420	5190	4530	3600	2790	2170	1630	1350
60,000	6520	6200	5440	4280	3340	2580	1950	1610
70,000	7600	7230	6350	4970	3900	3040	2280	1880
80,000	8700	8250	7260	5680	4470	3440	2600	2150
90,000	9800	9300	8130	6400	5020	3880	2930	2420
100,000	10,850	10,300	9070	7180	5580	4320	3250	2690

Subgrades from Zone C

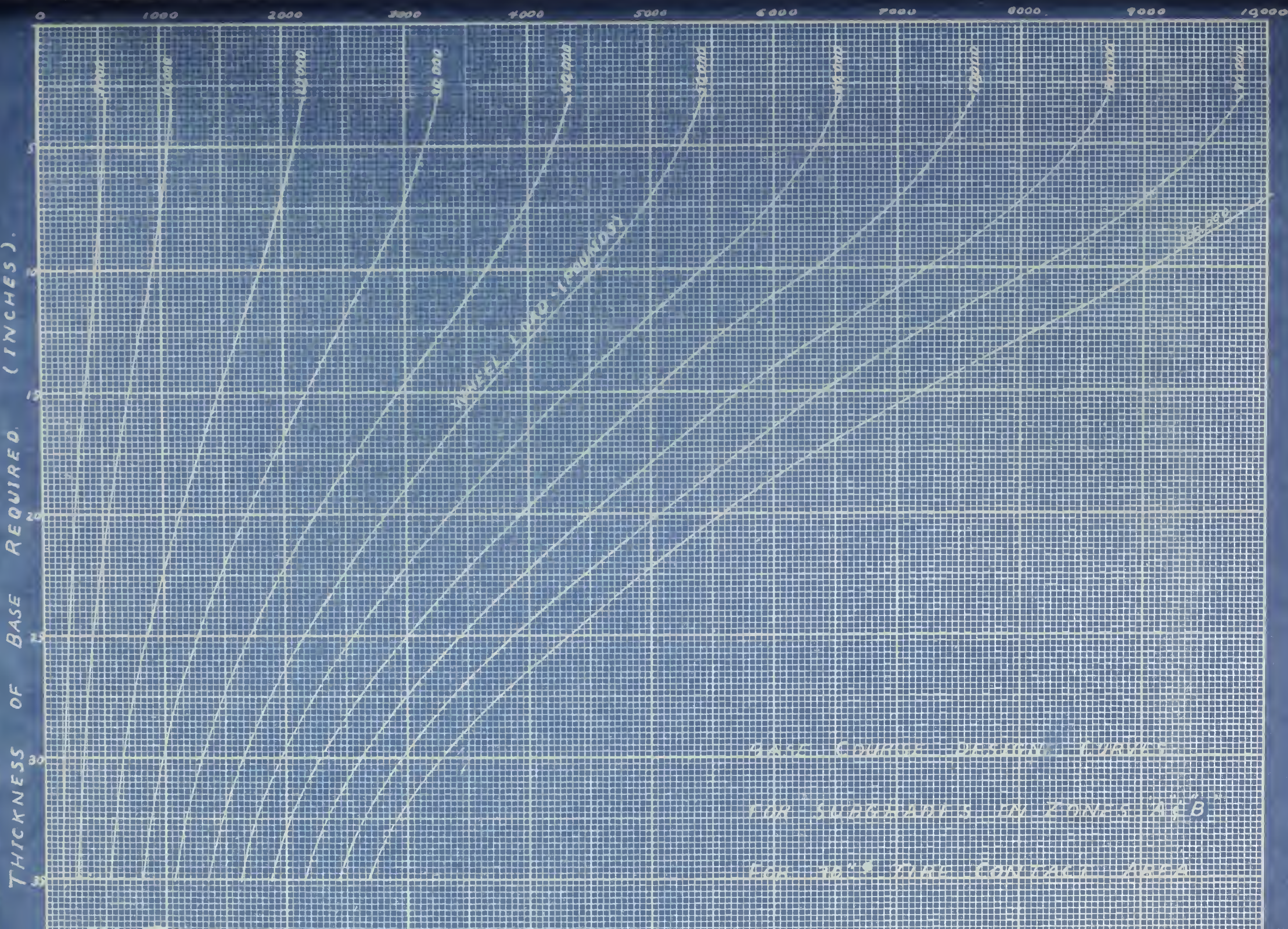
$$M_{0.05} = \frac{P \times C}{3.88}$$

P	$M_{0.05}$ for z =							
	3	6	10	15	20	25	30	35
2,000	280	265	232	184	143	111	93	68
5,000	695	662	532	460	358	277	209	173
10,000	1390	1320	1160	920	715	554	417	344
20,000	2780	2650	2320	1840	1430	1110	833	690
30,000	4180	3970	3490	2730	2150	1660	1250	1030
40,000	5570	5300	4650	3780	2860	2220	1660	1380
50,000	7050	6630	5820	4610	3570	2770	2090	1730
60,000	8350	7940	6980	5460	4280	3320	2500	2060
70,000	9710	9280	8130	6360	5000	3880	2920	2410
80,000	11,200	10,600	9300	7280	5720	4430	3340	2750
90,000	12,500	11,900	10,400	8200	6430	4980	3760	3100
100,000	13,900	13,200	11,600	9200	7150	5540	4160	3440

Subgrades from Zone D

$$M_{0.05} = \frac{P \times C}{5.37}$$

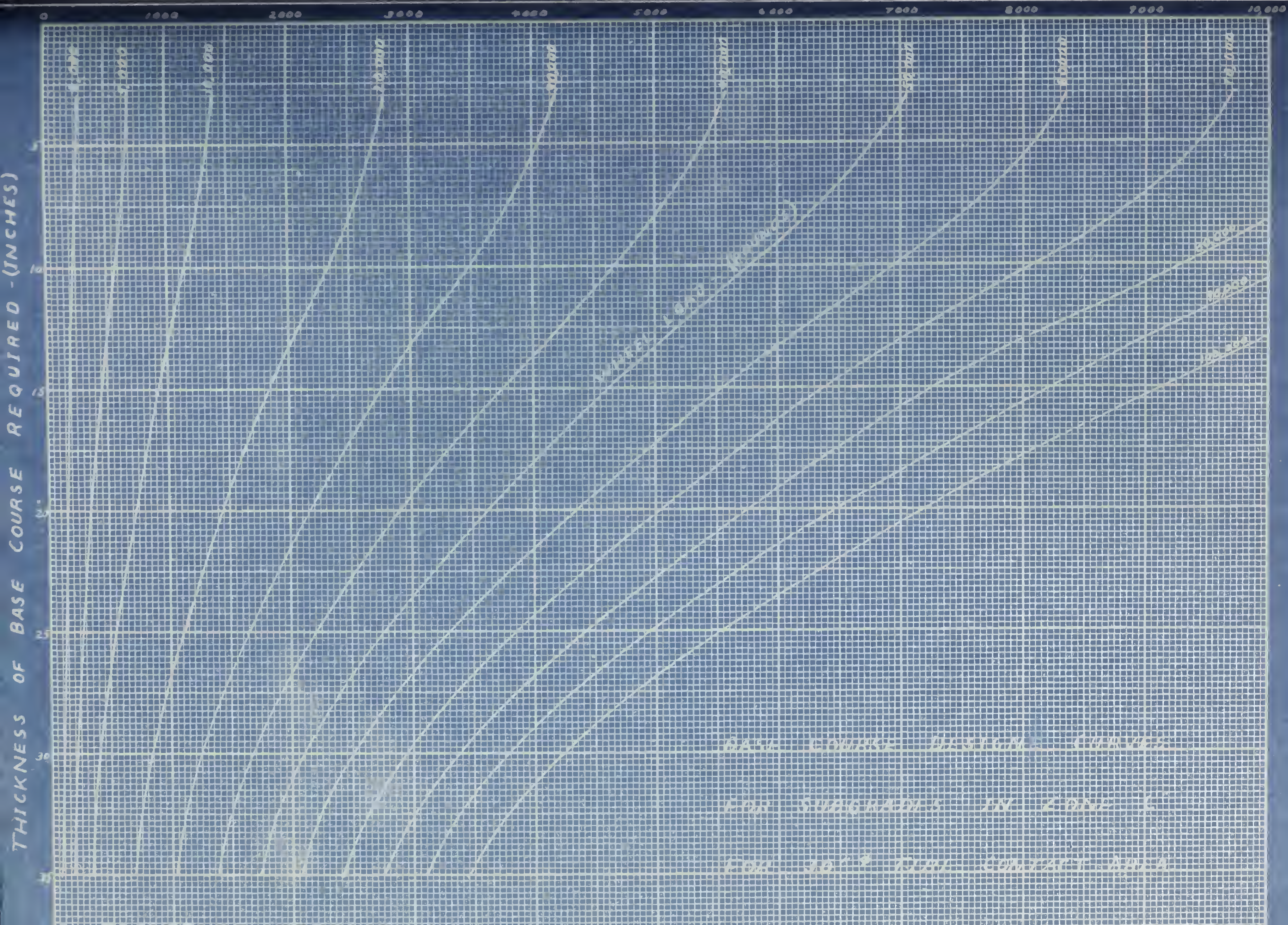
P	$M_{0.05}$ for z =							
2,000	306	279	254	201	157	121	91	75
5,000	760	725	636	503	391	304	228	189
10,000	1520	1450	1270	1010	782	607	455	376
20,000	3060	2900	2540	2020	1560	1210	910	753
30,000	4570	4350	3780	2980	2350	1820	1360	1120
40,000	6100	5800	5100	4130	3120	2420	1820	1510
50,000	7610	7250	6360	5040	3910	3040	2280	1890
60,000	9120	8690	7630	5980	4690	3630	2730	2250
70,000	10,600	10,200	8900	6950	5470	4250	3190	2640
80,000	12,200	11,600	10,200	7950	6250	4840	3650	3010
90,000	13,700	13,000	11,400	8960	7040	5440	4110	3390
100,000	15,200	14,500	12,700	10,100	7810	6060	4550	3760

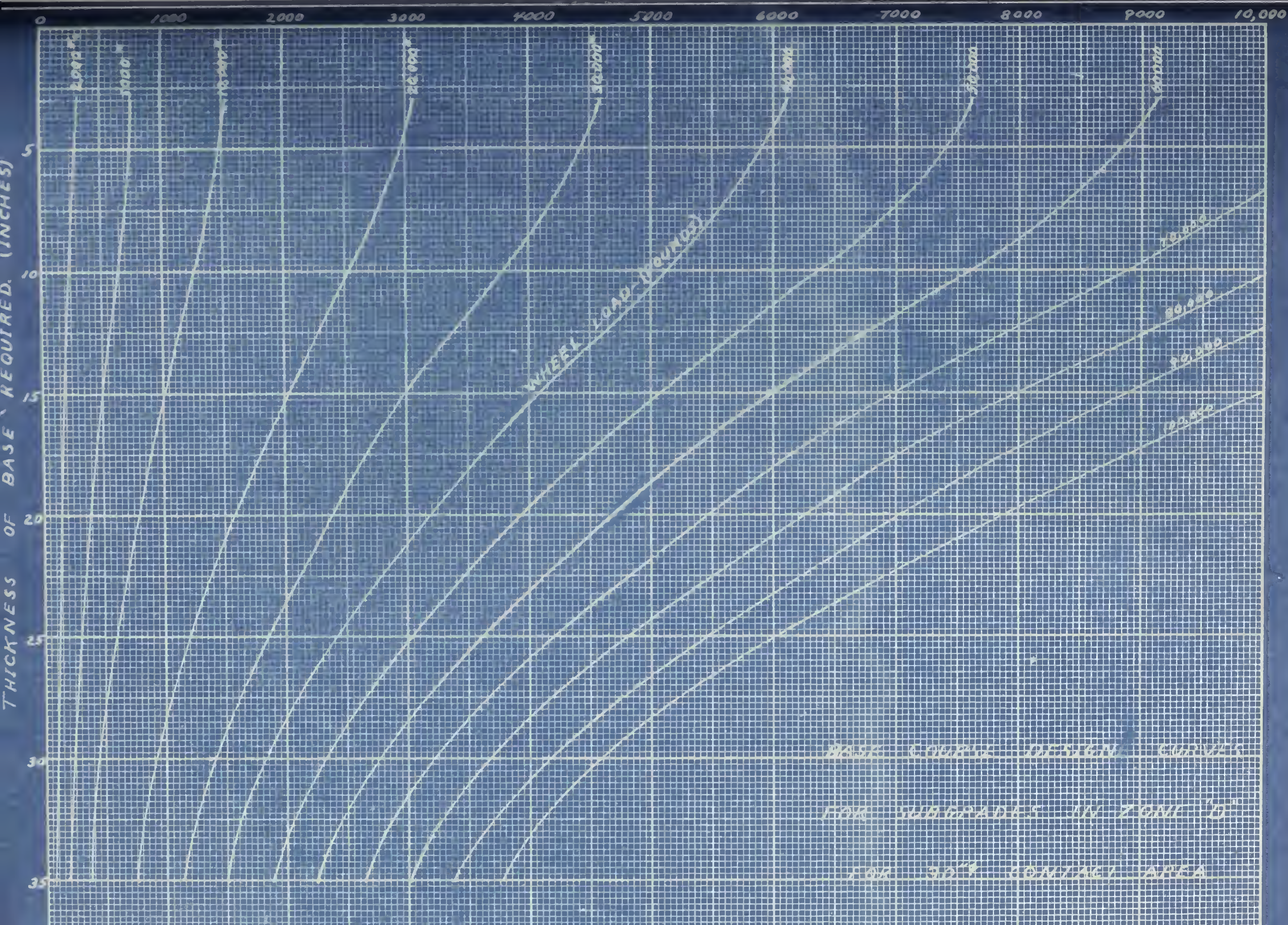


BASE COURSE DESIGN CURVES

FOR SUBGRADES IN ZONES "A" & "B"

FOR 10" TIRE CONTACT AREA


BASE COURSE DESIGN CURVES
FOR SUBGRADES IN ZONE C
FOR 30 TIRE CONTACT AREA
WHEEL LOAD (POUNDS)
THICKNESS OF BASE COURSE REQUIRED -(INCHES)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10,000
5
10
15
20
25
30
35


BASE COURSE DESIGN CURVES
FOR SUBGRADES IN ZONE "D"
FOR 30□" CONTACT AREA

The accuracy of these design curves can be checked by comparing the thicknesses obtained from the curves with the thicknesses that actually exist in the field, for wheel loads as obtained from the Base Course Load Test.

Subgrade from Zone	P	$M_{0.05}$	Actual z inches	Computed z inches
A	93,700	9540	5	7
	6,700	9630	7	0 (5)
	7,800	7650	5	7
B	47,500	3840	6	11
	32,800	4140	5½	0 (5)
	19,500	1920	6	7
	35,500	4140	6½	0 (5)
	34,300	2540	5	14
	40,200	4240	6½	5½
C	34,100	5080	7½	4 (5)
	48,300	7760	8	0 (5)
	50,000	5440	6½	11½
	52,500	6260	7	9
	35,000	3960	8	10
	34,000	3840	11	10½
	88,000	7760	16	16
	88,100	7620	16½	16
D	30,700	4210	9	8
	30,800	3950	9½	9½
	34,000	4140	10	10½

In no case in airport runway design would less than about 5 inches of material be placed above the subgrade. Thus if 5 inches is inserted for the computed values of z, wherever the value of "z" is less than this figure, it is seen that these charts as a whole give rather good checks with what was actually found in the field. The best agreement is seen in the subgrades from Zones C and D which indicates that probably the design curves for these subgrades are fairly accurate.

There is a wider discrepancy in the checks for the subgrades from Zones A and B, but even so, they give reasonably accurate results. Possible sources of error are as follows:

1. Value of bearing capacity of runway as found from Base Course Load Test. Very likely using the load on the plate required to produce a deflection of $\frac{1}{2}$ inch is not the most accurate method of finding the strength of a runway. The deflection at which failure occurs may vary with different types of subgrade and different types of base.
2. Values of modulus of deformation from subgrade load test. The error here should not be great. However, in practice, the modulus of deformation would be obtained from the Triaxial Test. Thus if representative samples were not acquired large errors could come in.
3. There is likely some error in the assumptions regarding distribution of stress at different depths. The extent of this error is not known.

This work of course has only been done for fire contact areas of 30 inches in diameter. If other diameters of tire contact areas were to be designed for, similar design curves would have to be made.

This work is done for the thickness of base course. It has been assumed that the surface material is as strong as the base; therefore the design curves give base plus surface thickness. This assumption is likely in order as a well constructed surface is, except possibly on very hot days, as strong or stronger than the base.

A comparison will now be attempted between the 5 different formulae or methods that were compared by Dr. McLeod * and the method proposed by the author in order to show the variation in results obtained.

In his comparison Dr. McLeod makes 3 assumptions.

1. That the California Bearing Ratio for any subgrade soil is approximately equal to the actual bearing capacity of the soil. It is a mere coincidence that this relation holds but it apparently does.**
2. That the value of "m", the shearing resistence of the base course in the Housel formula, is 35 p.s.i. an average value proposed by Professor Housel.
3. That the Klinger formula holds for compacted granular base courses as well as for hot mix asphalt pavements.

In his comparison Dr. McLeod made up tables for the different thicknesses of base plus surface required by each of the 5 methods of design for varying values of C.B.R. of the subgrade material. One of the tables was for a wheel load of 60,000 pounds and a radius of tire contact of 16 inches. The radius of tire contact used in this investigation was 15 inches. Therefore a reasonable comparison can be made between the 5 published methods of design and the method proposed in this paper.

The value of C.B.R. used in this comparison is that obtained from tests run on the samples in their natural state. An average value of the results obtained from the tests run at each depth at each location was used.

For the method proposed by the author values of "M averaged" from the triaxial compression test, for a lateral pressure of 15 p.s.i., were used.

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* Journals of Asphalt Technology (March-June 1943 & September-October 1943) by N. W. McLeod.

** Hogentogler - Engineering Properties of Soil. Page 227.

WHEEL LOAD 60,000#

Subgrades from Zone	Station	Thickness of Base Plus Surface Required in Inches					
		Housel	Klinger	California	Gray	Goldbeck	Author's
C	17+20	15.9	14.4	32	44	66	16
	23+00	13.9	8.7	17	23	37	19
	29+00	15.7	13.3	28	39	60	18
D	23+14	16.3	18.1	36	57	86	24
	19+07	15.7	13.5	28	33	61	20
	3+27	15.7	13.1	27	38	59	22

In the above table the radius of tire contact area for the Housel, Klinger, California, Gray and Goldbeck formulae is 16 inches while that for the author's method is 15 inches. If the author's values were based on a 16 inch radius of tire contact area the values of base course thickness would be slightly less.

An examination of this table indicates that the California method and the Gray and Goldbeck formulae give thicknesses of base course that are too high while the Housel and Klinger formulae lead to under design.

If the values of base course thickness obtained by the Housel and California methods are averaged, it is observed that the value obtained is very nearly the same as that obtained by using the author's method.

It is the opinion of the author that the method of design proposed here is superior to the methods that are now in use. It does however, require a little better laboratory and a trained man to run the tests and interpret the results.

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In conclusion a model design of an airport runway will be attempted to demonstrate the use of this method of design.

Suppose at some location south of Edmonton a new runway was to be constructed. This runway is to be 1 mile long, 200 feet wide and runs north and south. The topography is such that the north 1/3 mile is to be cut, the middle 1/3 mile is to be grade and the south 1/3 mile is to be fill. For this problem assume the cut is all to be 2 feet, and the fill is all to be 3 feet.

Commonly all the area to be covered by the runway should be stripped of all lean and deleterious materials.

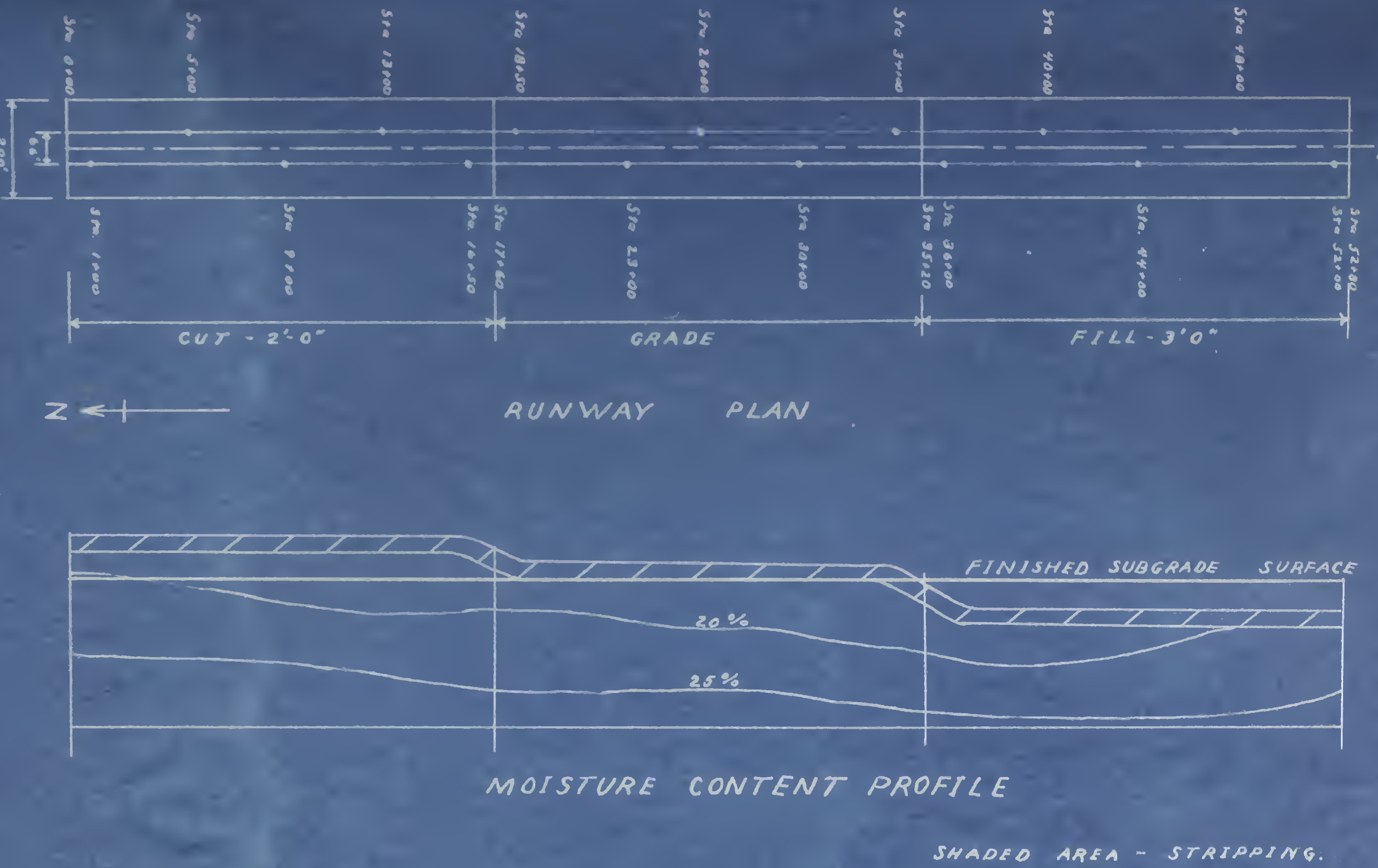
This will likely amount to 6 to 18 inches here. Assume 12 inches.

Sampling should be done at a time when the subgrade material is about as wet as it would be in the worst condition when under the runway. This would probably be early Spring in this area.

For a runway this size it would be advisable to sample along two lines, say along a line parallel to the centreline, 33 feet west of it and one 33 feet east of it. The first thing to do would be to establish elevations along these lines at approximately every 300 feet. Thus, knowing the grade of the runway, the finished elevation of the surface of the subgrade at each point could be determined. At each of these points where the elevations had been determined auger holes would be bored to about 8 to 10 feet below the proposed subgrade surface. This augering should be done under the direction of a competant man who could roughly classify the soils and take moisture samples. Moisture samples which would give the average moisture content of each foot of depth should be taken and moisture content determinations run. A moisture profile and soil classification profile of these auger holes along each line on the runway would be drawn and from these one can roughly determine where he should take samples.

Assume for this example that the soil classification and moisture content profile is nearly constant over the entire runway area. In such a case the sample locations could be rather uniform.

A possible outline of sampling locations along with a moisture content profile for one of the lines of sampling locations is shown in Figure 40. It would be unnecessary in this case to sample the portion on which the fill would be placed as the fill material would determine the design. However, the locations that should be sampled were it necessary have been shown.


Sta 0+00
Sta 5+00
Sta 13+00
Sta 18+50
Sta 26+00
Sta 34+00
Sta 40+00
Sta 48+00
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200'
66'
Sta. 1+00
Sta 9+00
Sta. 16+50
Sta 17+60
Sta 23+00
Sta 30+00
Sta 35+20
Sta 36+00
Sta. 44+00
Sta 52+00
Sta 52+80
CUT - 2'-0"
GRADE
FILL - 3'0"
N
RUNWAY PLAN
FINISHED SUBGRADE SURFACE
20%
25%
MOISTURE CONTENT PROFILE
SHADED AREA - STRIPPING.

FIG. 40.

At each sampling location in the cut and grade sections of the runway, the following samples should be taken.

20 pounds of the subgrade material between depths of 0-6 inches and also 9-15 inches below the finished subgrade surface elevation.

One 6 inch diameter by 6 inches long undisturbed sample from each of the same depths as the above 20 pound samples were taken.

In this case there is a 3 foot fill, so the tests would have to be run on the borrow pit material. If there were only 6 to 12 inches of fill one set of tests would be run on the borrow pit material and another set would be run on the first 6 inches of soil immediately below the stripping and design results obtained by interpolation. Assuming here the borrow pit material is constant in character, about 300 pounds of the representative material would be sufficient.

Liquid Limit, Plastic Limit and Plasticity Index tests would be run on the soils from all locations and thus by going to the zoned classification chart one can pick which zone his material is from.

Assume the soil from the test locations in the cut and grade districts is from Zone C, and that the soil from the borrow pit is from Zone A.

Triaxial tests would be run on the undisturbed samples taken from the cut and grade sections. Samples of the borrow pit material would be made up (preferably in no smaller than 6 to 8 inch diameter molds) to the same moisture content and density as expected in the field. Triaxial tests would now be run on samples taken from these. About 5 or 6 tests to get average value of "M" would be sufficient.

For each test location the values of Modulus of Deformation (M) as derived from the Triaxial Tests for the different tests would be averaged as previously explained.

Suppose the averaged Triaxial data is as follows and you are designing for a wheel load of 50,000 pounds for 30 inch diameter tire contact area.

Station	Offset	Average M p.s.i. from Triaxial Test	$M_{0.05}$	Thickness
1+00	33' W	2,430	5,200	12½"
5+00	33' E	2,550	5,400	11½"
9+00	33' W	2,430	5,200	12½"
13+00	33' E	2,600	5,500	11"
16+50	33' W	2,600	5,500	11"
18+50	33' E	3,000	6,100	8"
23+00	33' W	3,050	6,200	8½"
26+00	33' E	3,160	6,400	7"
30+00	33' W	3,100	6,300	7½"
34+00	33' E	3,000	6,000	8½"
Borrow Pit Material		1,250	4,600	10"

The values of $M_{0.05}$ are obtained by entering the charts of "M from Triaxial Test" vs "$M_{0.05}$ from the Field Test", for soils from the proper zone, with the value of average M from the Triaxial Test.

The values of thickness are obtained by entering the "Design Charts" for soils from the proper zone with values of $M_{0.05}$ as derived above.

From these results it would probably be recommended that 12 inches of base plus surface be placed on the cut area, 8 inches of base plus surface on the grade area and 10 inches of base plus surface on the fill area.

In practice the results may not come out as close as these and then it may be necessary to vary the thickness of base plus pavement, from one end of one section to the other.

If the results for design thickness varied considerably it may be necessary to carry on more sampling and testing on the runway to safely establish the design thicknesses.

It must be remembered that the design curves for thickness submitted here are of necessity only theoretical and have not near enough checks from actual tests to be sure that the curves are correct. However, these curves are a close approximation of the proper ones, as verified by the checks that have been obtained but should not be used for design until further verified by additional information.

Acknowledgements

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The Department of Transport for so kindly making available to the author the vast amounts of data used in this program.

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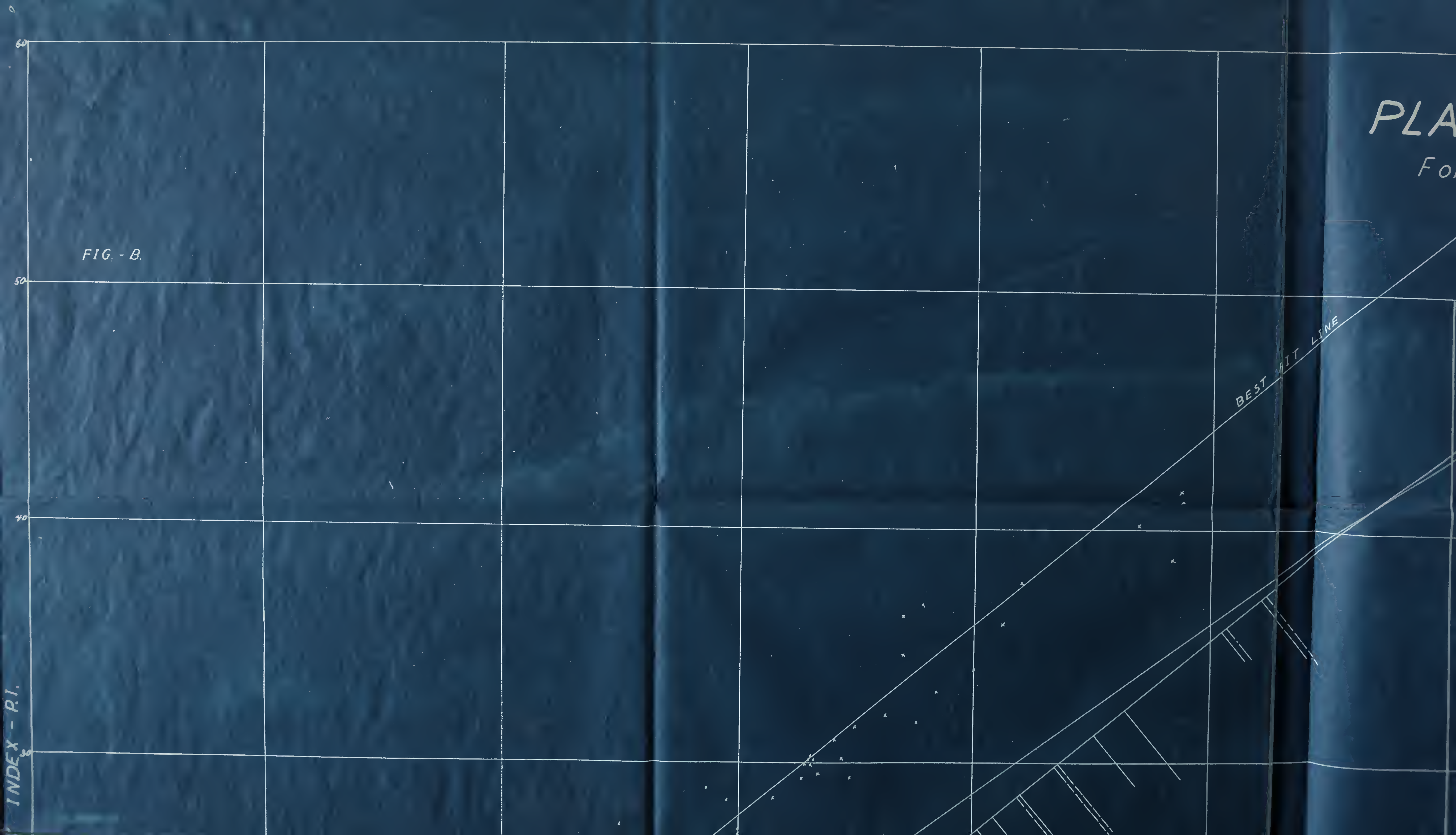
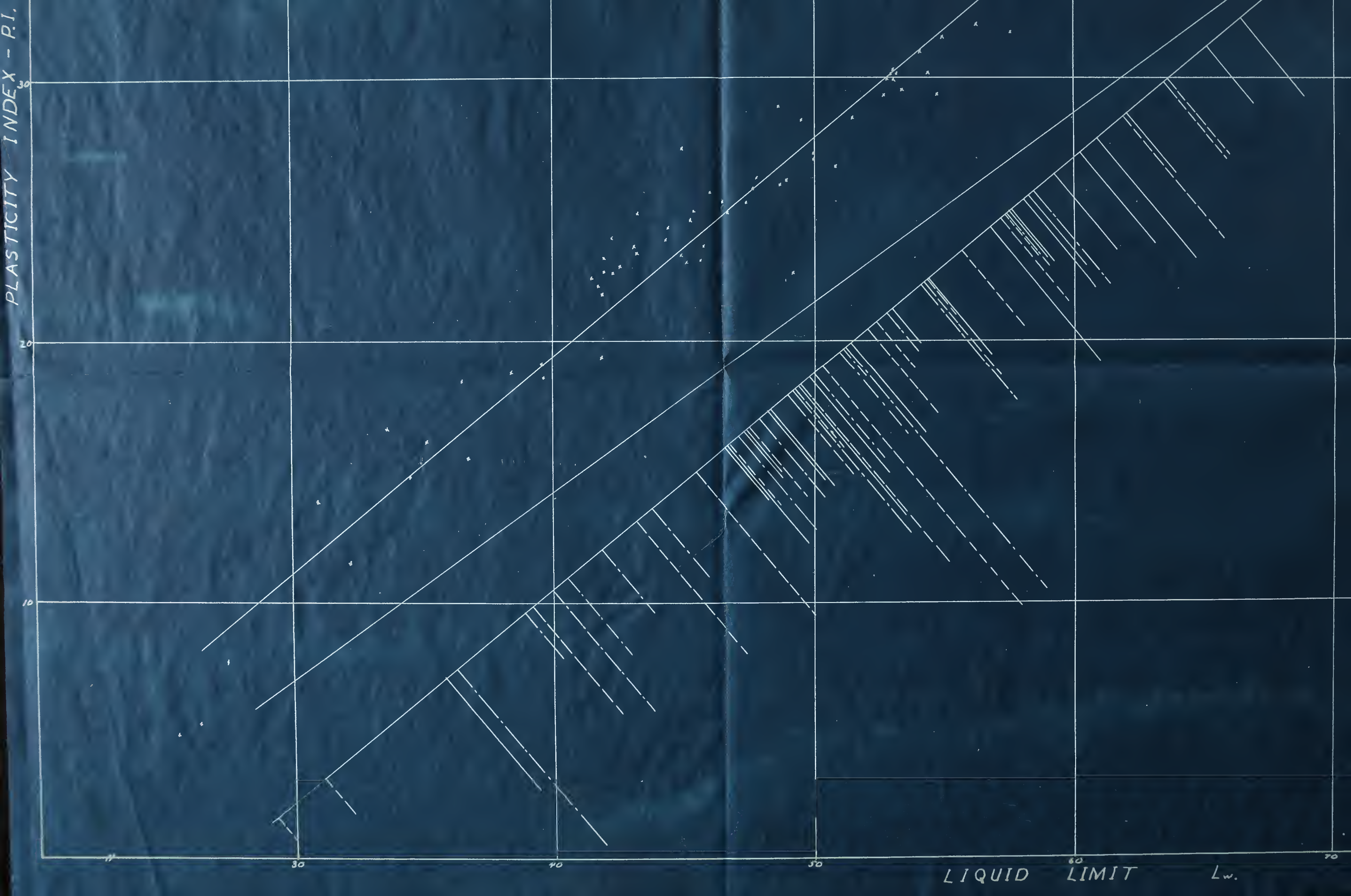
FIG. - B.



PLASTICITY CHART.

FORT ST. JOHN B.C.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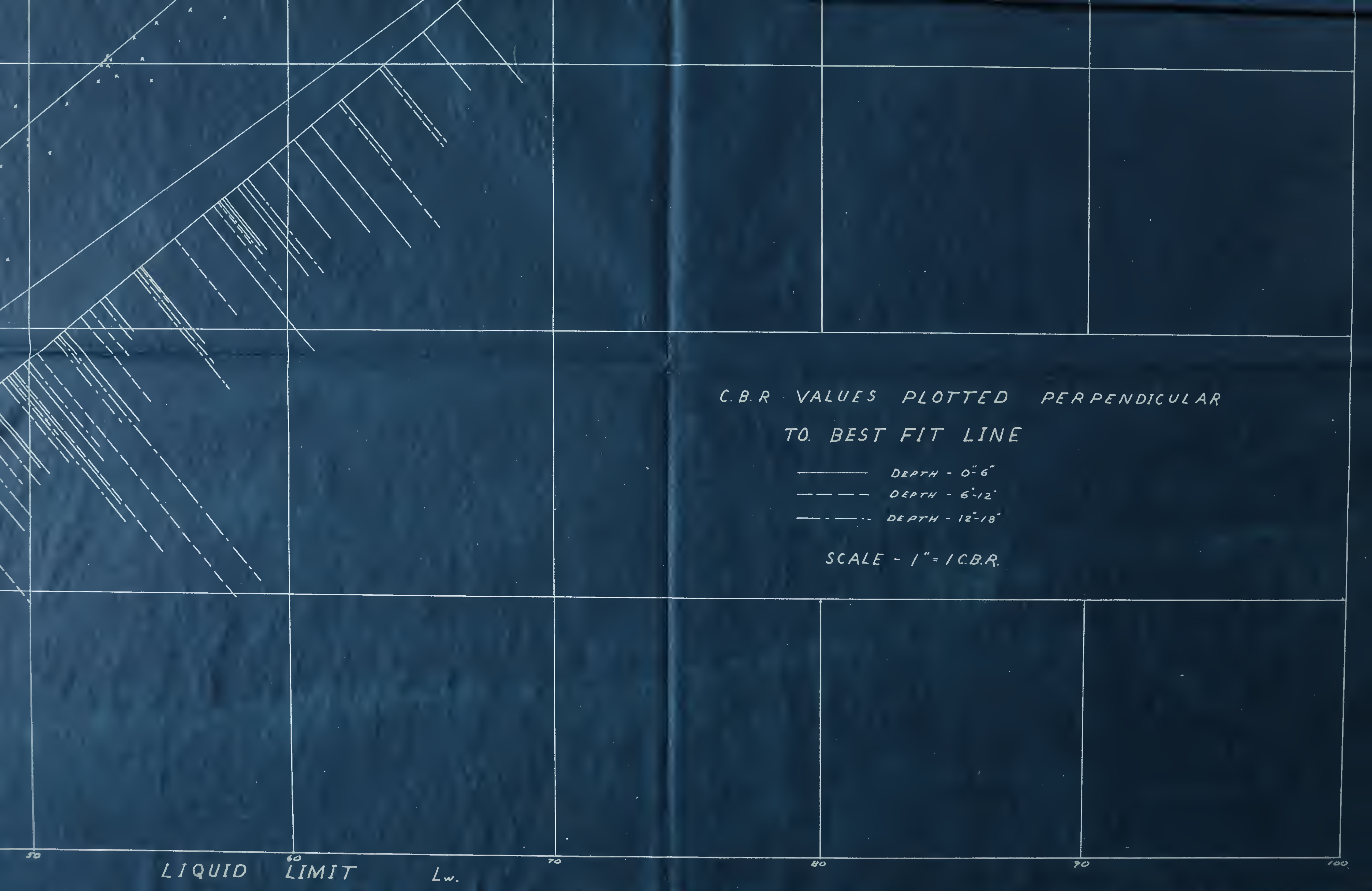
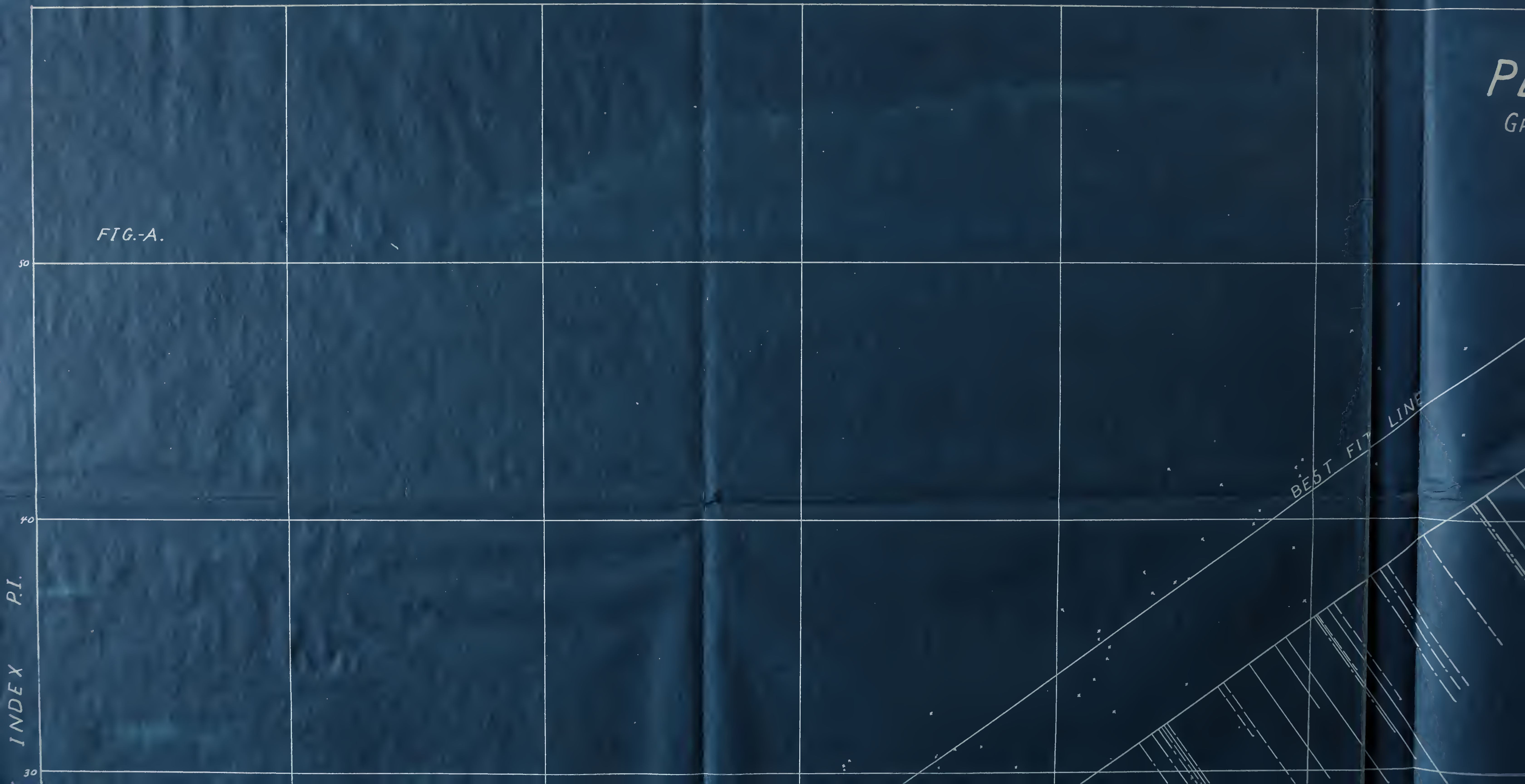
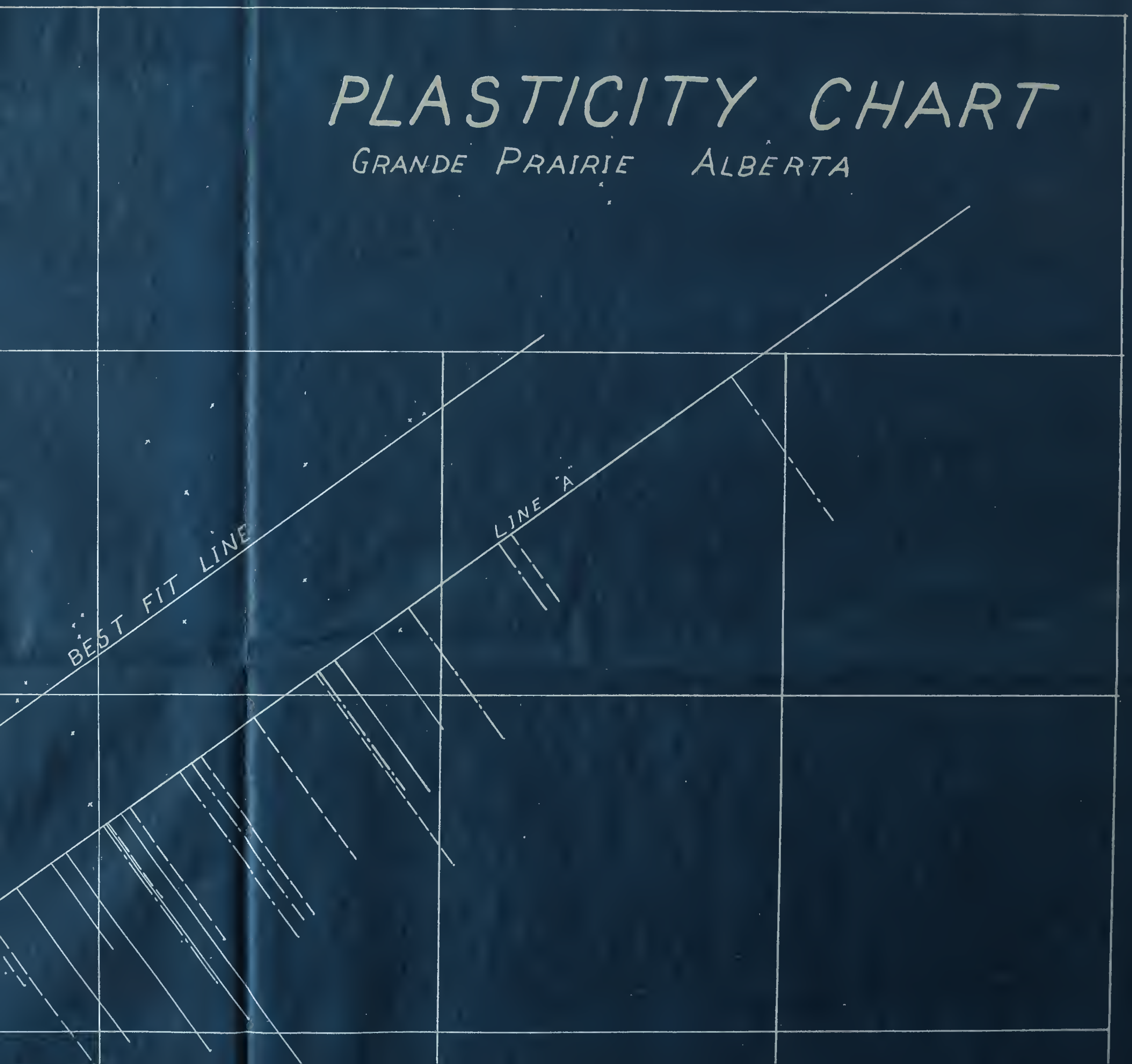
30
20
10
30
40
50
60
70
LIQUID LIMIT Lw.



C.B.R VALUES PLOTTED PERPENDICULAR TO BEST FIT LINE

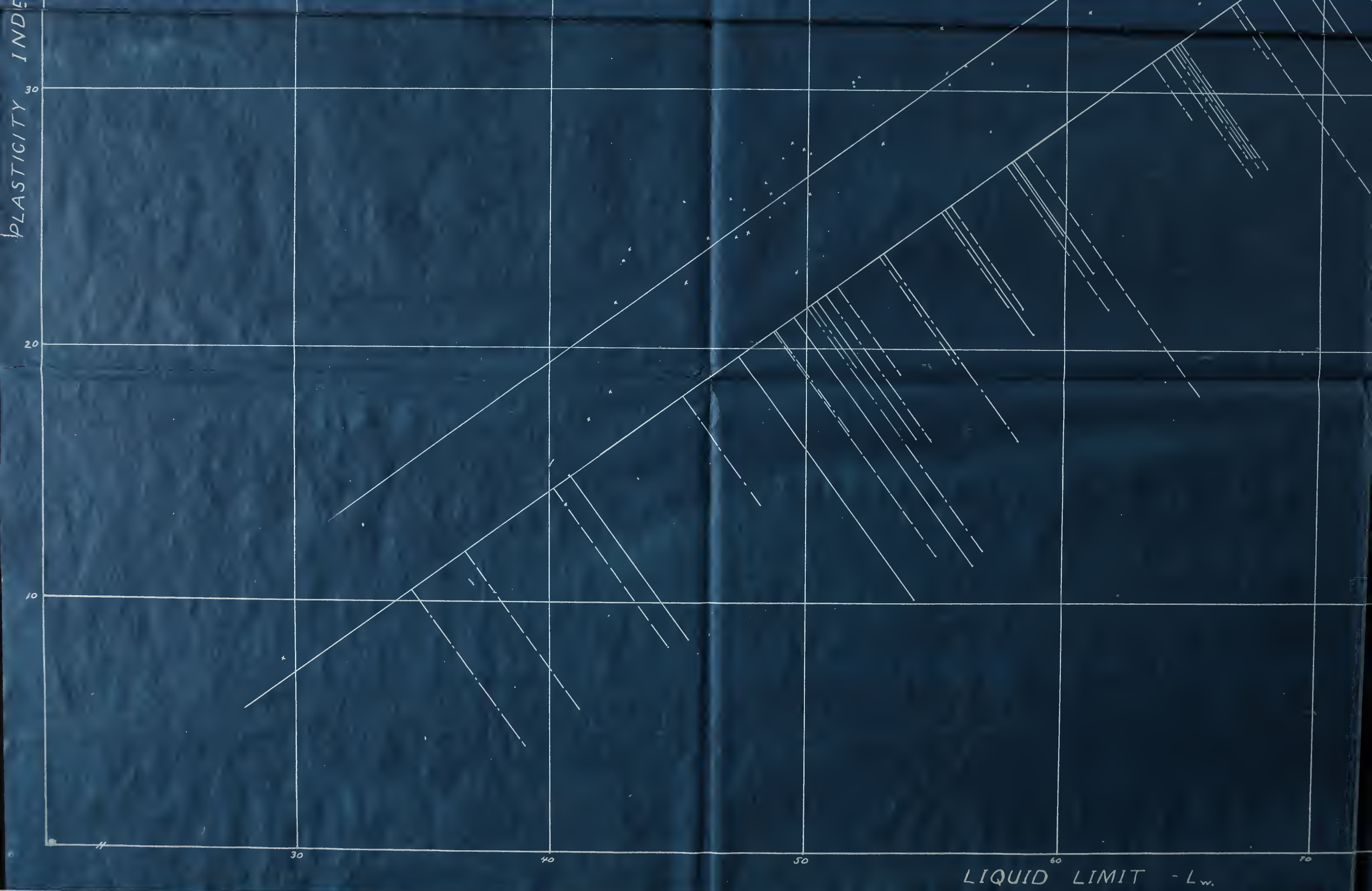
FIG.-A.

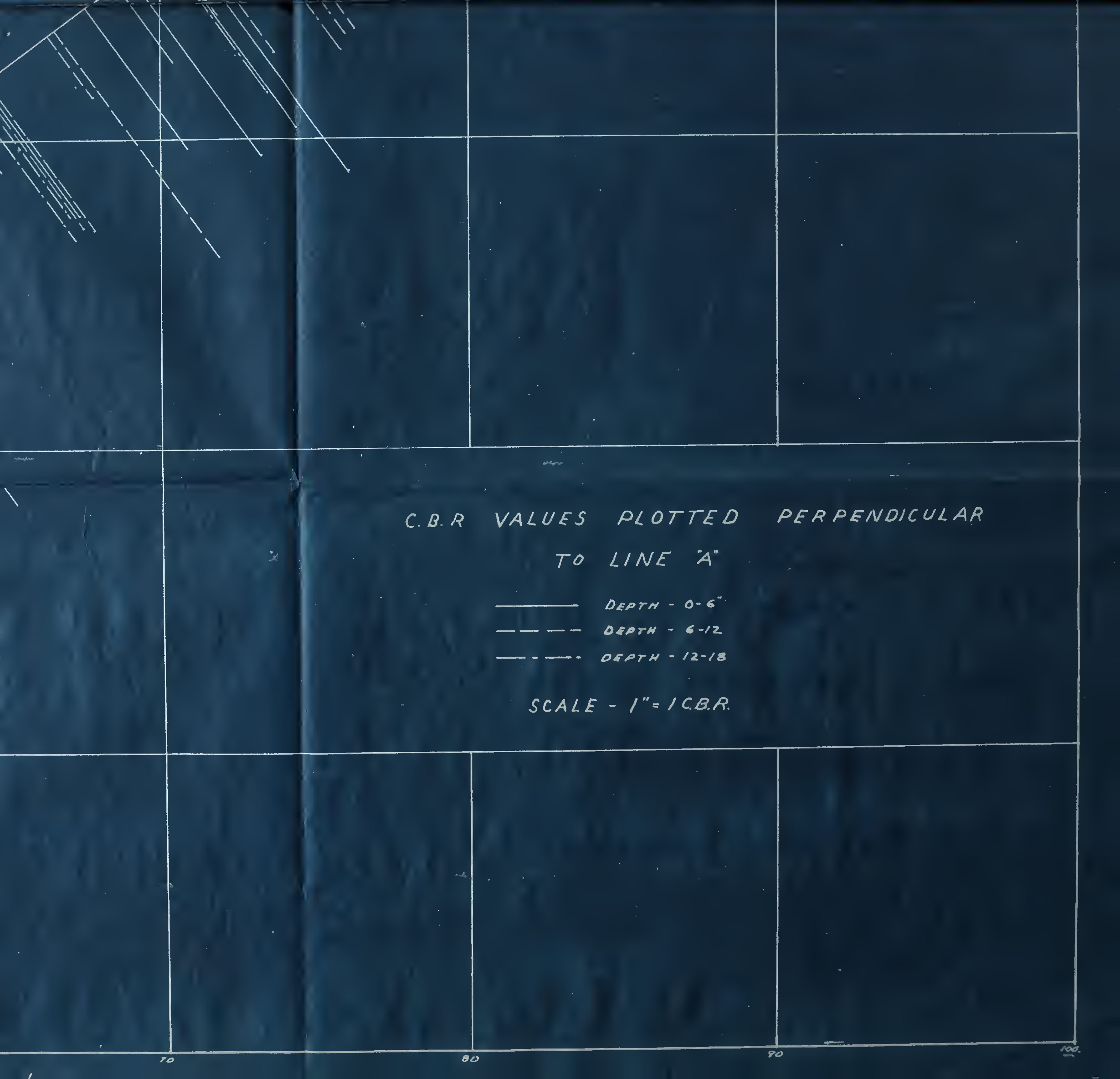




PLASTICITY CHART

GRANDE PRAIRIE ALBERTA


PLASTICITY INDE
30
20
10
30
40
50
60
70
LIQUID LIMIT - Lw.



C.B.R VALUES PLOTTED PERPENDICULAR TO LINE "A"

B29752